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INCLUDING PRESTRESSED CONCRETE

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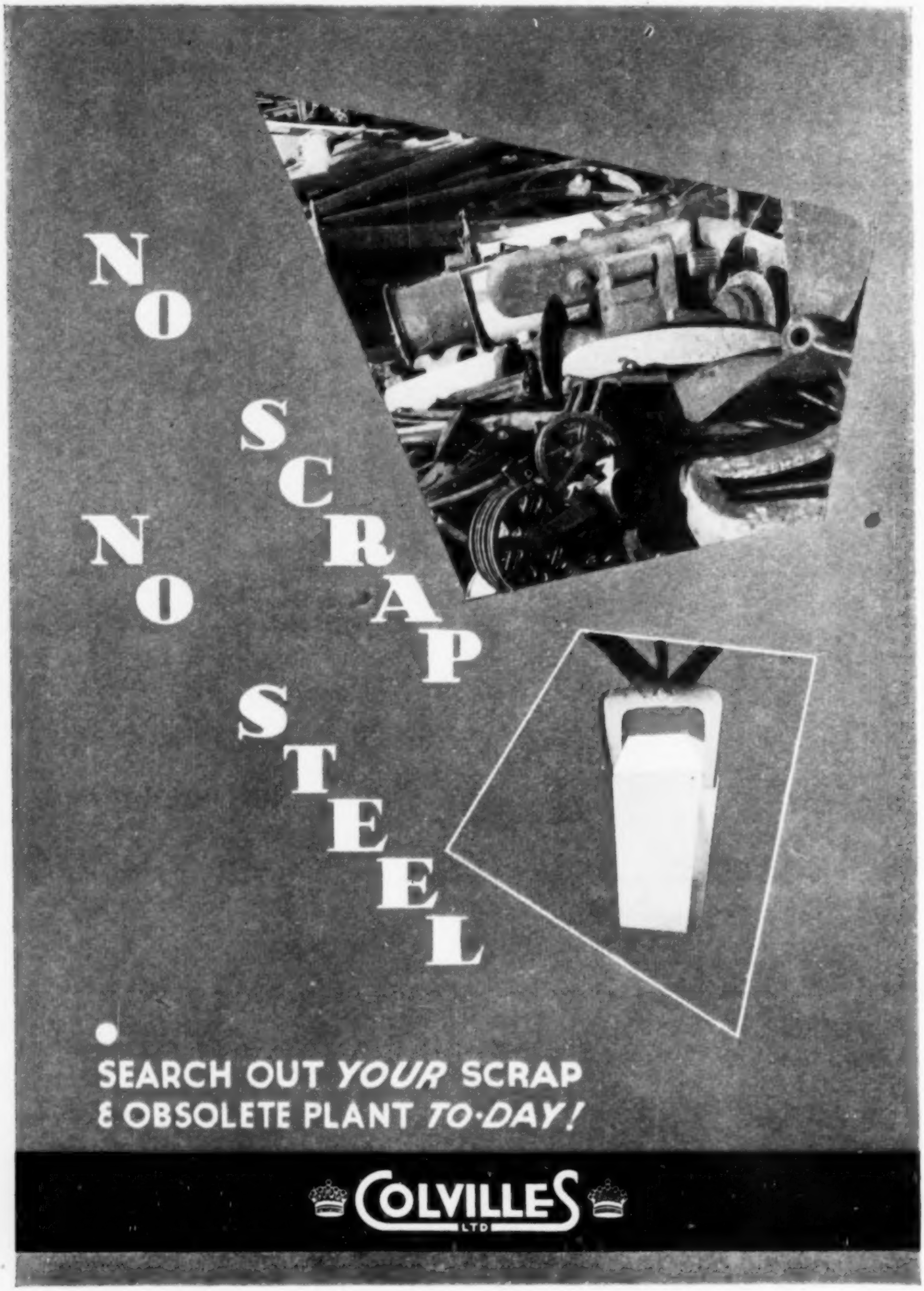

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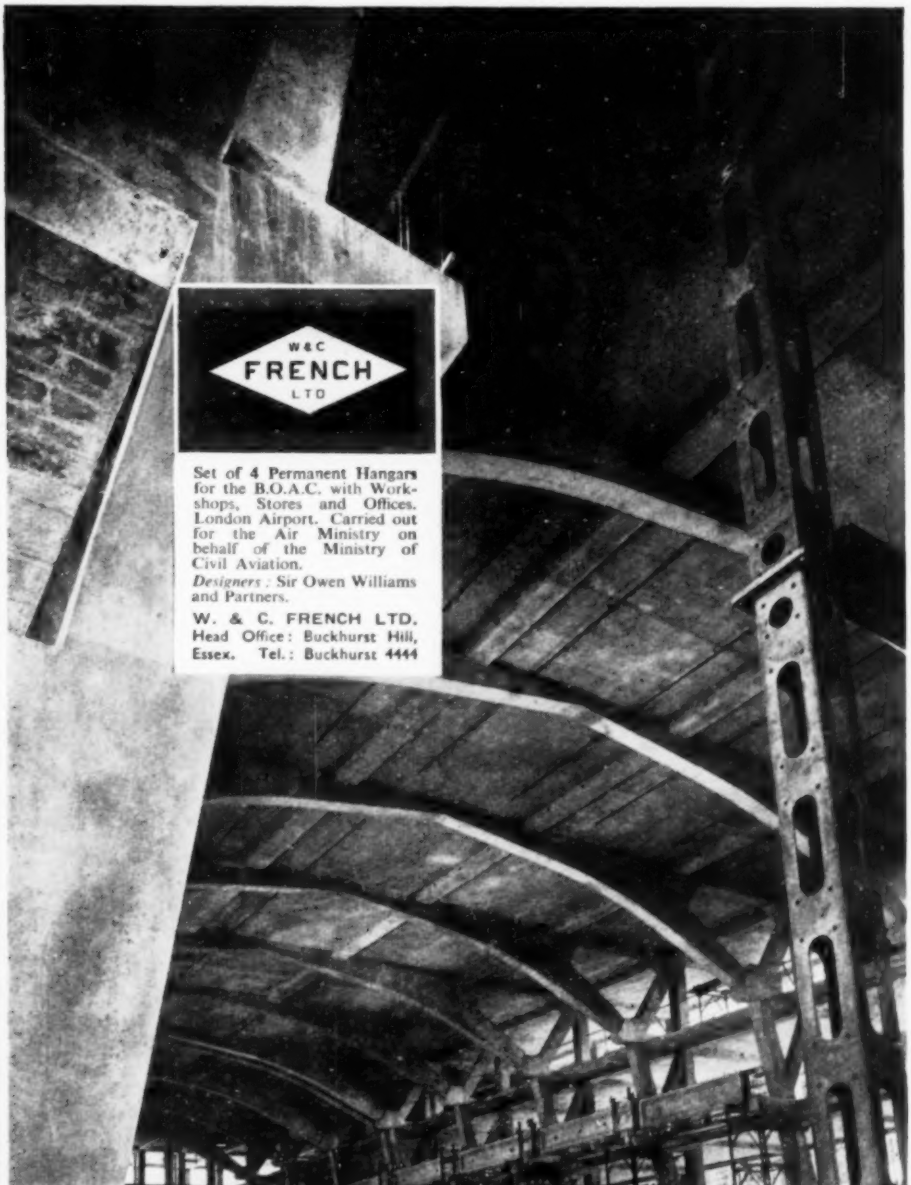
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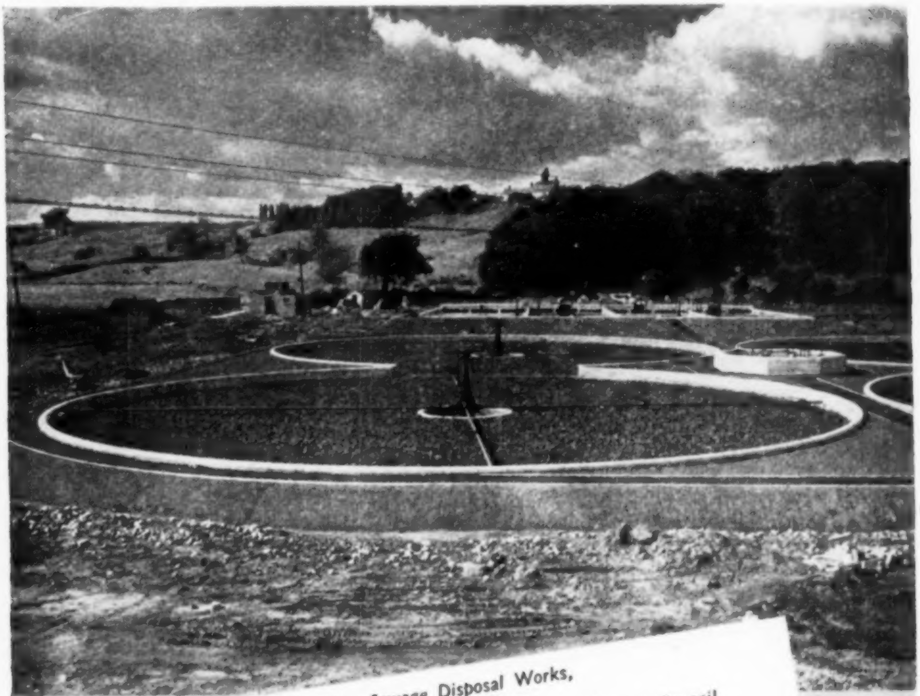

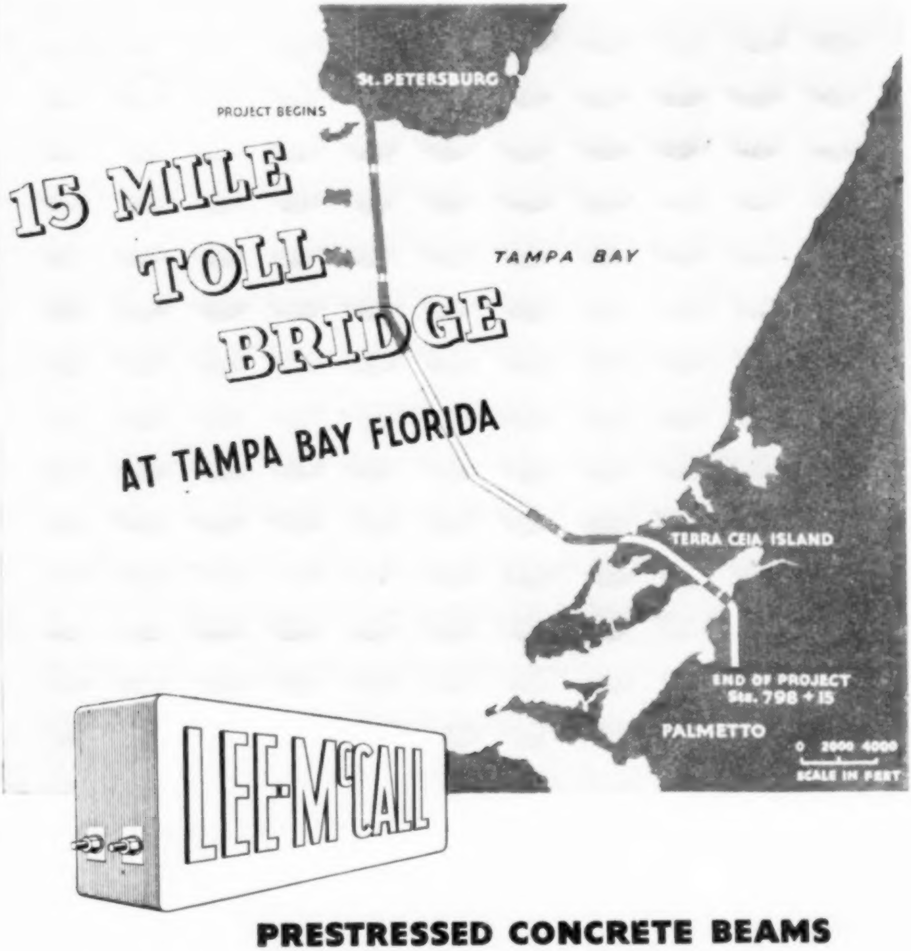

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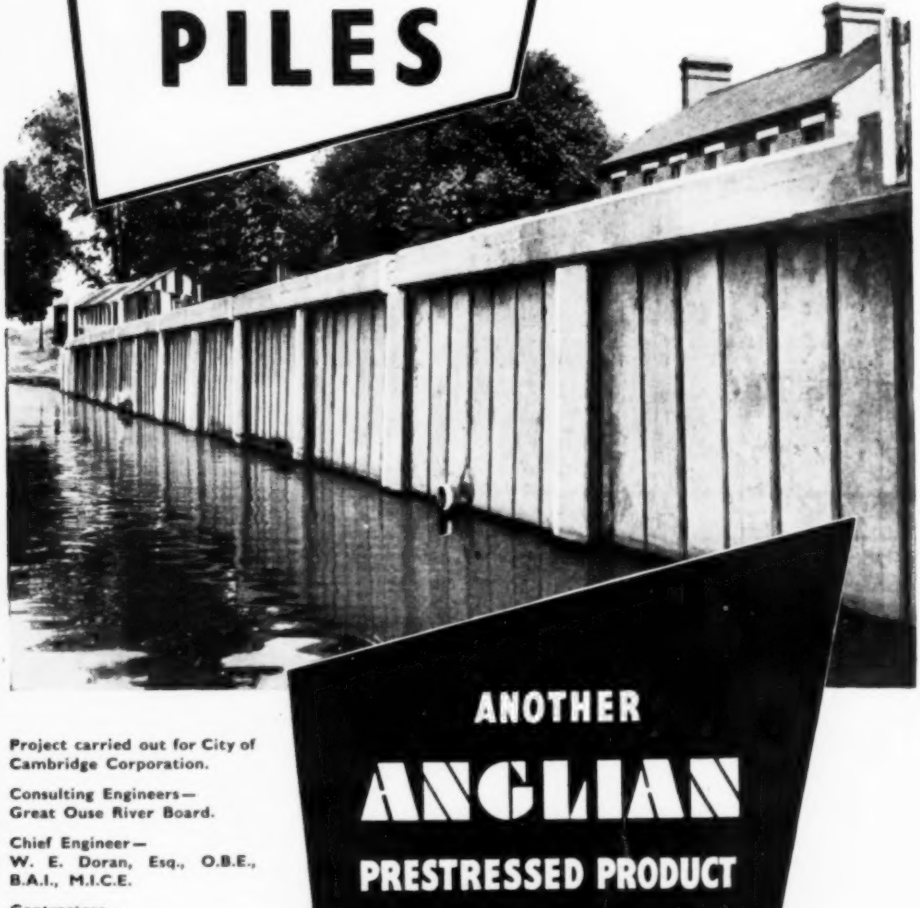

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EDITORIAL NOTES

Degrees.

IN our December, 1952, number we referred to the widespread and increasing use of designatory letters, especially those which signify that a person has passed an examination or has merely paid a fee. A correspondent now writes to suggest that the title "doctor" has in recent years become so common that its value is in danger of becoming debased.

In the first decade of the present century there were very few technical, as distinguished from medical or clerical, doctors in this country. The holder of a technical doctorate was invariably a man of distinction in his profession and was generally entirely engaged in teaching at a university. It is only in recent times that there has been any ambiguity about the designation "doctor". In former times, when divinity and the law were the principal subjects of study in the universities, the title was virtually limited to leading scholars of the church and of the law. Thus in the seventeenth century Isaac Barrow, whose mathematical discoveries were an inspiration to Newton, his more famous pupil, had the title doctor because he excelled in theology. Later the designation was extended to medicine. In this country we are not yet committed to such resounding titles as "Professor Baurat Doktor-Ingenieur", which may be applied to a man who may well be undistinguished except in one narrow branch of science or technology, and we have not yet followed the American custom of giving the title assistant professor to some of the university lecturers. To-day, however, following the large number of D.Sc. and Ph.D. degrees awarded at our universities and university colleges during the past thirty years, the word doctor has almost completely lost its former significance. Earlier in this century universities which had not previously granted Ph.D. degrees fell into line with the practice of some foreign universities. The reason for this decision may have been partly an economic one, that is, it may have resulted from a desire to attract students from abroad and thus add to the income of the university and of the nation. There were, of course, other advantages. Post-graduate instruction became a recognised and valuable part of the work done at the universities, and science and industry have reaped the benefit of research carried out in pursuit of higher degrees.

In using the title which they have received, doctors of technical subjects are following the custom of the church and of medicine. Generally the foreign student

wishes to have this title to take home to his own country, where perhaps there are fewer doctors of medicine and no doctors of divinity with a long-established priority of right to use this prefix. At present there is a very large number of foreign students in this country, but this is not a guarantee that they will continue to come here when the affairs of the world are more settled. In the year 1938 we asked a Professor from Tokyo Imperial University why it was that there were many fewer Japanese students in London than there were ten or fifteen years previously. His answer was that they were attending German universities. The cost of sending their sons to a European university was, he explained, a heavy burden on the parents, and they preferred the German university because there the students spent longer periods at lectures and had shorter vacations. In fact, the parents thought that they got better value for their money. Also, perhaps because they worked harder or perhaps because degrees were more freely awarded, a greater proportion of the students got a higher degree in Germany than in this country. Standards of examinations have been seriously lowered in some cases in recent years; indeed, it was stated by the educational correspondent of "The Times" recently that it is now no more difficult to get some degrees than it was fifteen years ago to get a Schools' Certificate at the age of 16½ years. There is also a tendency to class as scientists people who are not in fact scientists. Three years ago a committee appointed by the Government stated that nearly 5000 newly-qualified "scientists" were being produced every year. This was the number of people awarded science degrees at the universities, and their classification as scientists ignored the possibility that many of these young people would not continue scientific pursuits at all, or would be unfitted for anything but routine work. Indeed the report referred to the "output" of scientists as though scientists and the scientific type of brain could be produced by a machine.

Our correspondent suggests that the prefix doctor might be restricted to technical doctors who have achieved a position of renown in their profession. At present a man may call himself doctor who has neither renown, eminence, responsibility, nor even competence except in one very narrow field of learning. A person might, for example, be a doctor of some branch of applied mathematics with no knowledge of anything connected with practice, or he might be a doctor of laboratory experiments with no experience in design. By all means let the universities continue to reward patience and accuracy, diligence and application, but it is suggested that the time has come to consider the wisdom of permitting the indiscriminate use of a prefix which is now subject to so many interpretations as to be almost meaningless. This is not to say that there must be no exceptions. Not every doctor of science or philosophy need be debarred from calling himself doctor. Every year the names of a few notable men in science and engineering are awarded honours by the Sovereign, and in almost every case these honours are a reflection of public esteem. If a similar procedure were adopted in conferring the active title of doctor, the prefix would be limited to men of distinction and responsibility. One suggested solution is to limit the use of the prefix to recipients of honorary degrees and to let the technical doctorate obtained by the submission of one thesis lie in the records as evidence that the holder has satisfied the examiners in a particular subject. The eighteenth century provides a precedent. Dr. Johnson did not get a degree while he attended a university, but was awarded an honorary degree when his work had proved his worth.

The Ultimate Strength of Prestressed Beams.

By PROFESSOR G. MAGNEL.

It is not yet possible to calculate exactly the greatest bending moment which a prestressed beam can resist, although several writers have attempted to do so. In some theories, however, certain coefficients appear to be introduced so as to make the formulæ agree as nearly as possible with the results of experiments, which moreover are sometimes made on beams which are too small to give results of real value. It must be remembered that for two beams of different cross sections but of the same span and to carry the same load, and designed by the elastic theory for the same working stresses, the factor of safety may be very different. In the writer's opinion this problem should be solved with the least

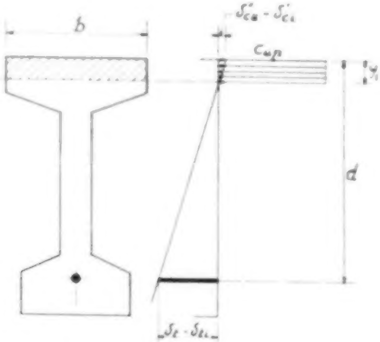


Fig. 1.—Distribution of Stresses at Failure.

possible calculation, as calculations are based on assumptions which may lead to wrong results. It is therefore proposed to use known experimental results to produce a reasonable formula, avoiding the temptation to confuse the problem with pseudo-scientific frills.

An analysis of the rupture of beams shows that two possibilities exist :

(1) If the steel has a cross-sectional area A_T less than about 0·2 per cent. of the product bd, failure occurs mostly by the breaking of some of the wires before the top concrete is crushed. However, before failure the concrete is widely cracked so that the active section is reduced to the area by_1 and the steel area A_T (*Fig.* 1). As the concrete is not far from being crushed in the compressive zone, it may be assumed that the stress is uniform. Hence the lever arm of the cross section just before failure is $d - \frac{y_1}{2}$, and the moment causing failure is $M_t = A_T t_u\left(d - \frac{y_1}{2}\right)$, where t_u is the ultimate resistance of the wires and M_t the moment when the steel fails. In all cases known to the writer where the percentage of steel is between 0·1 and 0·2 of the area bd, the value

of y_1 is so reduced in comparison with the depth that the error cannot be as much as 10 per cent. assuming that

$$M_t = 0{\cdot}95\, A_I t_u d \quad . \quad . \quad . \quad . \quad . \quad (1)$$

(2) When the area of steel is greater than 0·2 per cent. the concrete is generally crushed and the steel does not break. However, here also a large crack occurs before failure and the stress distribution of *Fig.* 1 can still be assumed, with a larger value of y_1 compared with the first possibility ; generally the part left in compression is entirely in the top flange of the beam ; it is recommended that the dimensions of a beam be chosen so as to be sure that this is so (see later). The moment at failure is in this case $M_e = by_1c_{up}\left(d - \frac{y_1}{2}\right)$, where c_{up} is the ultimate crushing resistance of concrete prisms. This is the point where attempts are often made to calculate the value of y_1, for example by adopting the law of plane deformations, which permits one to write, by pure geometry,

$$\frac{\delta'_{eu} - \delta'_{ei}}{\delta_t - \delta_{ti}} = \frac{y_1}{d - y_1}$$

in which δ'_{eu} is the ultimate strain of the compressed concrete, δ'_{ei} is the strain of the top concrete due to prestressing, δ_t is the strain of the steel at failure of the beam, and δ_{ti} is the strain of the steel due to prestressing. But such a formula is useless as we ignore the value of most of these strains. It is simpler to write $M_e = Kbd^2c_{up}$ and to use a value of K as found from experiments, remembering that K is theoretically $\alpha\left(1 - \frac{\alpha}{2}\right)$ with $\alpha = \frac{y_1}{d}$. The value of α (and consequently K) varies mainly with the ratio of steel, that is $\lambda = \frac{A_T}{bd}$.

Unfortunately there are few experimental results of which all the data are available ; it is necessary to know the value of b, d, λ, and c_{up} in order to be able to calculate K. In most publications one of these values is not given, or the beams are too small to give useful results.

Tables I and II give all the required values for eight beams, classified in the order of increasing percentages of steel. These beams, except No. 1, failed due to crushing of the concrete.

In *Fig.* 2 are plotted the values of K as ordinates, the ratio being the abscissæ, and it is seen that the eight points are generally in a straight line. The values of $K_1 = Kc_{up}$ are also plotted, giving $M_e = K_1bd^2$, and the eight points obtained are even more in a straight line, the equation of which is $K_1 = 214{,}000\lambda$ (in lb. per square inch). Consequently

$$M_e = 214{,}000\lambda bd^2 \quad . \quad . \quad . \quad . \quad . \quad (2)$$

Formula (2) applies only to the following cases, which are the only ones considered in establishing it :

(a) Where the ultimate strength of the wires is between 208,000 lb. and 248,000 lb. per square inch and where the stress in the steel during prestressing is between zero and 144,000 lb. per square inch.

(b) Where c_{up} is between 5180 lb. and 7550 lb. per square inch.

(c) Where d is between 10·5 in. and 70 in.

TABLE I.

No. of beam	b (in.)	d (in.)	A_T (in.²)	100λ	c_{up} (lb. per sq. in.)	M_e (in.-lb.)	K	K_1 (lb. per sq. in.)
1 *	15·75	28·70	0·73	0·161	5770	4,240,000	0·057	331
2	11·80	20·90	0·49	0·198	5770	2,230,000	0·075	431
3 †	11·80	10·42	0·28	0·227	5920	627,000	0·082	485
4	57·80	58·70	10·08	0·298	7550	131,000,000	0·087	653
5	52·00	70·00	15·25	0·420	5190	232,000,000	0·175	892
6	9·85	7·88	0·49	0·628	5840	768,000	0·215	1250
7	9·85	12·20	0·97	0.810	6200	2,620,000	0·287	1780
8	5·32	13·60	0·60	0·840	5330	1,670,000	0·318	1690

* This beam would fail by rupture of the wires.

† This beam is not prestressed, but reinforced with twisted rectangular strips of steel (known as Neptune steel) with the same properties and cross-sectional area as the round wires in the other beams.

TABLE II.

No. of beam	t_u (lb. per sq. in.)	$\frac{d}{b}$	Span l (ft.)	$\frac{l}{d}$
1	216,000	1·83	9·85	4·1
2	216,000	1·76	39·40	22·6
3	207,000	0·85	16·40	18·9
4	221,000	1·02	112·50	22·9
5	248,000	1·35	158·00	27·1
6	249,000	0·80	13·15	20·0
7	249,000	1·24	19·70	19·3
8	240,000	2·56	—	—

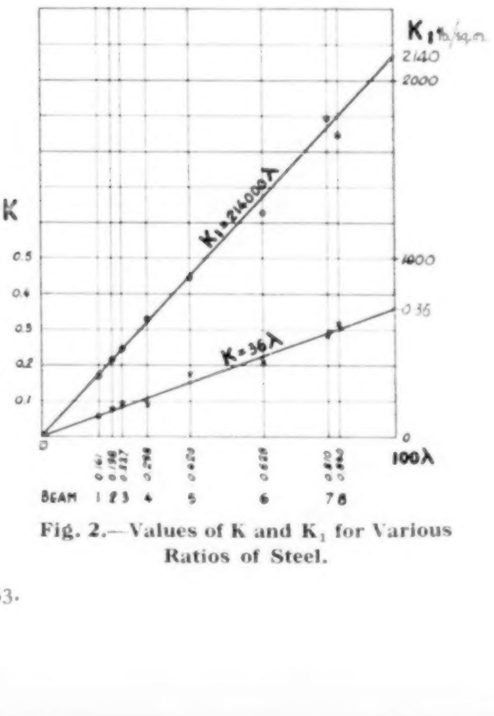


Fig. 2.—Values of K and K_1 for Various Ratios of Steel.

(d) Where λ is between 0·161 and 0·840 per cent.

(e) Where the cables are anchored or where wires with increased bond are used.

(f) Where the ratio of span to depth is between 4·1 and 32·2.

(g) Where $\frac{d}{b}$ is between 0·8 and 2·36.

(h) Where the thickness of the top flange is at least equal to the calculated thickness as a function of λ.

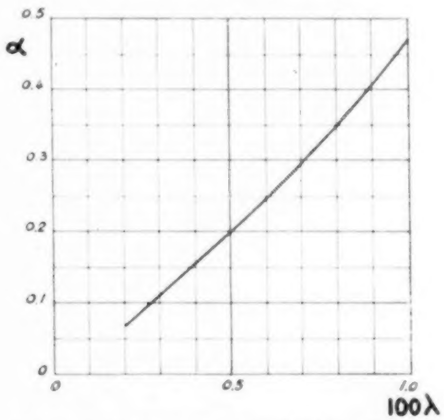


Fig. 3. Values of α for Various Ratios of Steel.

Most practical beams are included in these cases. Case (h) is equivalent to stating that the thickness of the top flange must be greater than y_1 at rupture, or αd.

Fig. 2 gives K as a function of λ with an average of $K = 36\lambda$. Hence $\alpha\left(1 - \frac{\alpha}{2}\right) = 36\lambda$, which gives a parabola for α as a function of λ (*Fig.* 3).

For the eight beams considered α can now be calculated, and hence the minimum thickness required for the top flange (*Table* III).

Beam No. 3 is not prestressed but simply reinforced with Neptune steel,

TABLE III.

No. of beam	100λ	α	Minimum thickness (in.)	Real thickness (in.)
1	0·161	0·060	1·73	3·94
2	0·198	0·074	1·68	3·94
3	0·227	0·100	1·02	3·94
4	0·298	0·131	6·70	7·88
5	0·420	0·185	11·20	11·50
6	0·628	0·276	2·05	rectangular
7	0·860	0·357	4·20	rectangular
8	0·840	0·370	5·03	—

which has high tensile strength and increased bond. This beam behaves like the others because the strain in the wires at prestressing is only about 0·5 in a thousand, and is negligible in comparison with the ultimate strain.

Beam No. 1 failed due to the wires breaking; notwithstanding this, it follows the same law as the others. This is because formula (1) can be written $M_T = 0{\cdot}95 t_u \lambda b d^2$, and $0{\cdot}95 t_u$ is about 214,000 lb. per square inch.

The following important conclusions may be drawn from the foregoing:

(a) In the beams considered, the ultimate moment is independent of c_{up}.

(b) The ultimate moment is proportional to λ.

(c) The cross-sectional area of the wires for a beam of width b and having to resist an ultimate moment M is proportional to d.

(d) If S is the factor of safety against rupture, we can find from formula (2)

$$d = \beta\sqrt{\frac{M}{b}} \text{ with } \beta = \sqrt{\frac{S}{214{,}000\lambda}} \quad . \quad . \quad . \quad (3)$$

where M is the total moment $M_p + M_a$. For example, if $S = 2{\cdot}5$, the following are the values of β as a function of λ (in lb. and in.):

For $100\lambda = 0{\cdot}3$	0·4	0·5	0·6	0·7	0·8
$\beta = 0{\cdot}0624$	0·0542	0·0483	0·0441	0·0410	0·0383

In cases where $\lambda = 0{\cdot}5$ per cent., and $M_p = M_a$ for a reinforced concrete beam and $0{\cdot}5 M_a$ for a prestressed beam, we have

$$d = 0{\cdot}0483\sqrt{1{\cdot}5}\sqrt{\frac{M_a}{b}} = 0{\cdot}0592\sqrt{\frac{M_a}{b}} \text{ with } A_T = 0{\cdot}000296b\sqrt{\frac{M_a}{b}}.$$

The same beam in reinforced concrete with working stresses of 1000 lb. per square inch in the concrete and 20,000 lb. per square inch in the steel requires

$$d = 0{\cdot}0738\sqrt{2}\sqrt{\frac{M_a}{b}} = 0{\cdot}1045\sqrt{\frac{M_a}{b}} \text{ with } A_T = 0{\cdot}001126b\sqrt{\frac{M_a}{b}}.$$

Hence the depth of the reinforced concrete beam is decreased by 1·76 and the area of the steel by 3·77. This economy increases as the span increases. The foregoing conclusions show the possibility of using the following design method for a beam to resist a given bending moment.

Assume that M, b, and S are given, together with the limiting total depth from which we can derive the effective depth d.

From formula (2), $\lambda = \dfrac{SM}{214{,}000bd^2}$. If d were not given, λ would be assumed and $d = \beta\sqrt{\dfrac{M}{b}}$, β being given by (3). In both cases the thickness of the top flange is given by $\alpha = 1 - \sqrt{1 - 72\lambda}$.

So far we have obtained the dimensions of the top flange, the values of d, and the area of steel A_T. We must now choose the width of the web by taking into account whether a cable has to be raised or not, but it is important to make the width large enough to allow for easy concreting of the beam.

The only unknown is now the dimension of the bottom flange. Its minimum thickness is determined by the cover required for the cables. To find the width of the bottom flange it is sufficient to try two or three values, checking each

time, by the elastic theory, to see if the stresses in the concrete are not too high. We must mainly concentrate on the stresses in the lower part of the beam; indeed in the top of the beam the maximum compressive stress is immaterial as it is known that there is a factor of safety against ultimate failure.

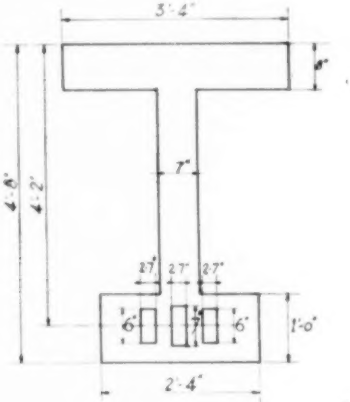


Fig. 4.

EXAMPLE (*Fig.* 4).—l = 98 ft. q = 1370 lb. per foot. b = 40 in. Factor of safety = 2·5. d = 50 in. From formula (3), assuming p = 950 lb. per foot, we calculate $\beta = 0{\cdot}0546$, $\lambda = 0{\cdot}39$ per cent., and $\alpha = 0{\cdot}154$. Consequently we require $A_T = 7{\cdot}8$ sq. in., say, 128 wires of 7 mm. (0·276 in.) diameter; this will be obtained by a middle cable of 48 wires and two side cables of 40 wires each.

The minimum thickness of the top flange will be $0{\cdot}154 \times 50 = 7{\cdot}6$ in., say, 8 in.

The dimensions of the cables require a thickness of the bottom flange of 12 in. A thickness of the web of 7 in. will allow for raising the central cable. It remains only to decide the width of the bottom flange. The minimum of 16 in. is not sufficient. A width of 28 in. gives, by the elastic theory, the following stresses:

		At prestressing (lb. per sq. in.)	In course of time (lb. per sq. in.)
Prestress + p	Top	392	484
	Bottom	2105	1615
Prestress + p + q	Top	1697	1789
	Bottom	1675	185

(It is assumed that the tensioning force is 144,000 lb. per sq. in. with $\eta_l = 0{\cdot}85$.)

The solution is shown in *Fig.* 4, although in practice this must be changed slightly by giving some slope to the lower surface of the top flange and the top surface of the bottom flange so as to make it possible to remove the forms.

Note.—After writing the foregoing the writer saw the result of a test on a large Freyssinet beam, and this agrees with the writer's formula within 3 per cent.

The Strength of Concrete at the Time of Loading.

PROFESSOR A. L. L. BAKER, Professor of Concrete Technology at the Imperial College of Science and Technology, City and Guilds College, London, has sent the following comment on the suggestion made in the Editorial Note in this journal for November, 1952.

"Since in a reinforced concrete structure the compressive strength of the concrete increases with age, it seems logical that the permissible compressive stresses should be related to the anticipated strength of the concrete at the time of application of the full load. However, the deviation from the mean of cube strengths at various ages, and also the variation of the mean strength with age for various mixtures and aggregates, require study in order that the factor of safety may remain about the same when a structure is loaded.

"If the working stresses are based on the strength at 28 days the concrete will invariably have a greater strength when it is loaded, because the load is seldom applied at 28 days. If, however, the working stresses are based on the strength at three months, it is almost certain that the full load will be then applied, and the increased strength we can now generally depend upon between 28 days and the time of loading will not be present. Therefore, while the stresses may be increased roughly in the same proportion as the known increases in the strength of the concrete, the ratio of the load causing failure to the working load should also be slightly increased.

"To achieve more consistent values of the load-factor, permissible working stresses also require increasing for other reasons, such as the greater strength of concrete which can be obtained with improved cements, better proportioned mixtures, and more thorough compaction."

Regulations for Reinforced Concrete of the London County Council.

ON January 1, 1953, new building by-laws were issued by the London County Council. These regulations, in so far as they relate to reinforced concrete, do not differ materially from British Standard Code of Practice No. 114, and the clauses relating to the dead and imposed loads are the same as those in British Standard No. 648 and Code of Practice No. 3, Chapter V, respectively.

Two qualities of concrete for reinforced concrete are specified, as in the regulations of the London County Council for 1938, namely, ordinary quality and quality A. The crushing strengths and working stresses for ordinary-quality concrete are as specified in 1938 and for quality A concrete are the same as those specified in the Code of Practice, as are those for high-alumina cement concrete. Subject to the approval of the district surveyor, the permissible compressive stresses may be increased by 10 per cent. for vibrated concrete if the crushing strengths are also increased by 10 per cent.

The allowable pressures on plain or reinforced concrete foundations, which are not specified in the Code are, in tons per square foot: For ordinary-grade concrete—1 : 1 : 2 mixtures, 50; 1 : 1½ : 3, 44; 1 : 2 : 4, 39; 1 : 6, 20; and 1 : 8, 15. For quality A concrete—1 : 1 : 2 mixtures, 96; 1 : 1½ : 3, 80; 1 : 2 : 4, 64; For high-alumina cement concrete—1 : 2 : 4 mixtures, 96.

The working stresses in steel reinforcement are the same as in the Code. The permissible stress in the reinforcement in axially-loaded columns is now 18,000 lb. per square inch, the modular ratio not being considered.

For slender columns the stress reduction coefficients are the same as those in the Code, but the ratio of the effective

length of a column to its least lateral dimension must not exceed 36. The clause in the Code which allows the maximum stress due to bending to be determined without reference to the reduction coefficients for sections within one-eighth of the column length from the centre-line of the beams is not included in the L.C.C. regulations.

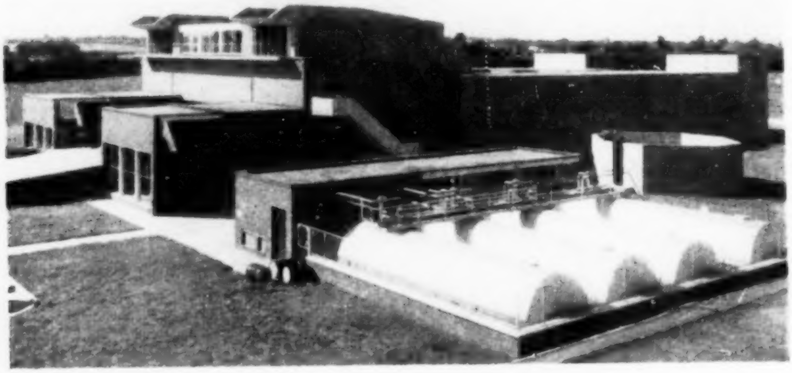
Where reinforced concrete is in contact with earth the minimum thickness of cover to the reinforcement is to be 3 in., unless special precautions are taken to prevent corrosion of the reinforcement.

The new regulations do not deal with prestressed concrete, but prestressed concrete may be used in buildings subject to the approval of the Council in each case.

No information is given regarding slabs reinforced in two directions or flat slabs, but it is understood that approval will be given to designs based upon the recommendations of the Code of Practice.

Copies of the new regulations may be obtained (price 3*s.* 6*d.*), through booksellers.

Jet-Engine Test-House and Exhaust Tunnels.



An interesting structure recently completed comprises reinforced concrete floors and a roof to a jet-engine test-house and ancillary departments. The long-span upstanding beams at the upper roof level are a feature of the work. These are supported by brick walls on plain concrete foundations, but deep reinforced concrete beams are provided where necessary to span large openings for air-ducts.

The two main exhaust tunnels are entirely in reinforced concrete and are 20 ft. wide by 126 ft. long by 18 ft. high. The tunnels are designed to resist internal and external pressures caused by accidental internal pressure or by variations in the temperature of the air. The instantaneous heating conditions are such that slag-wool protection is provided to the 1-ft. thick walls, which are hinged at their bases to allow movement due to temperature changes. A sliding surface was provided under the base-slab and a deep anchor-beam is employed to ensure that movement occurs from one end only and to prevent widening of the joints. The dog-legged section of tunnel housing the " silencing splitters " was designed to resist the component of a thrust of 25,000 lb. from the jet-pipe.

Additional reinforced concrete work includes an external cantilever staircase, the foundation and roof of the fuel installation and pump house, and an octagonal water tank of 20,000 gallons capacity.

The whole of the work was designed under the direction of Mr. Eric Ross, F.R.I.B.A., and the construction was carried out by Sir Alfred McAlpine & Sons, Ltd. The design of the reinforced concrete work was by the British Reinforced Concrete Engineering Co., Ltd., who also supplied the reinforcement. The structure is at the works at Filton of the Bristol Aeroplane Co., Ltd.

Prestressed Beams for a Bridge Three Miles Long.

A BRITISH SYSTEM USED IN THE U.S.A.

THE Lower Tampa Bay viaduct in Florida, U.S.A., is a toll road almost entirely over the open sea; part of the viaduct is hydraulic fill and the remainder a continuous bridge. Messrs. Pearsons, Brinckerhoff, Hall & MacDonald prepared the scheme as consultants in collaboration with Mr. W. E. Dean, chief bridge engineer of the State of Florida. This scheme included a high-level steel bridge at one part and reinforced concrete trestle-bridge construction aggregating nearly five miles. After the contracts had been placed for two short lengths of reinforced concrete trestle construction with spans of 36 ft., a number of firms were invited to submit alternative schemes in prestressed concrete. The Preload Company of New York in conjunction with McCalls Macalloy, Ltd., of Sheffield, submitted a design prepared by Mr. Donovan H. Lee, M.I.C.E., which was based on the use of the Lee-McCall system of prestressing with high-tensile alloy steel bars. The total length of bridge for which these alternative tenders were invited was slightly over 3 miles.

Three prestressed schemes, one using wire cables, another 0·2-in. diameter drawn wire, and the third the Lee-McCall system using high-tensile bars, were considered in comparison with the reinforced concrete design. The prestressed concrete design using high-tensile bars was the cheapest by 184,000 dollars, although the tender included the freight and import duty for the bars. The contract price for this part of the work was 8,000,000 dollars.

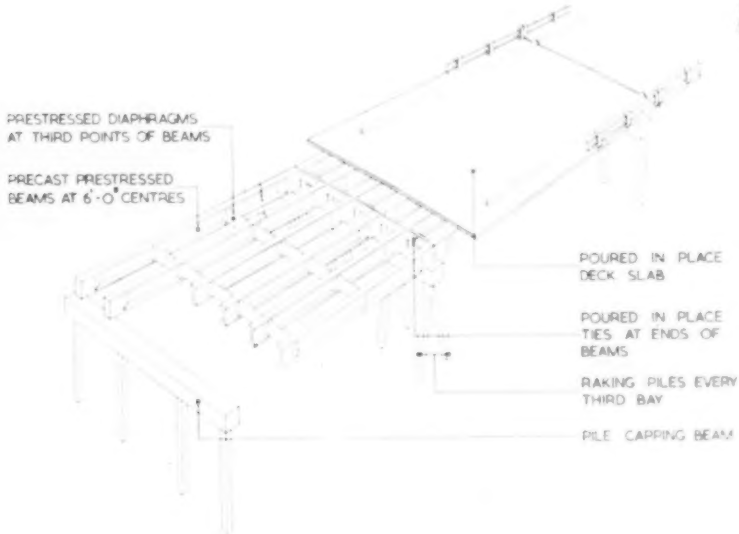


Fig. 1.—Isometric View.

The design uses trestles at 48-ft. centres and prestressed precast beams supporting an in-situ deck (*Figs. 1 and 5*). Six beams are used for each span and spaced 6 ft. apart under the roadway, which is of 28 ft. clear width with footpaths cantilevering beyond. In the reinforced concrete design with a smaller span there are four beams spaced 9 ft. 10 in. apart. The reinforced concrete deck is 6¾ in. thick in the case of the prestressed concrete bridge

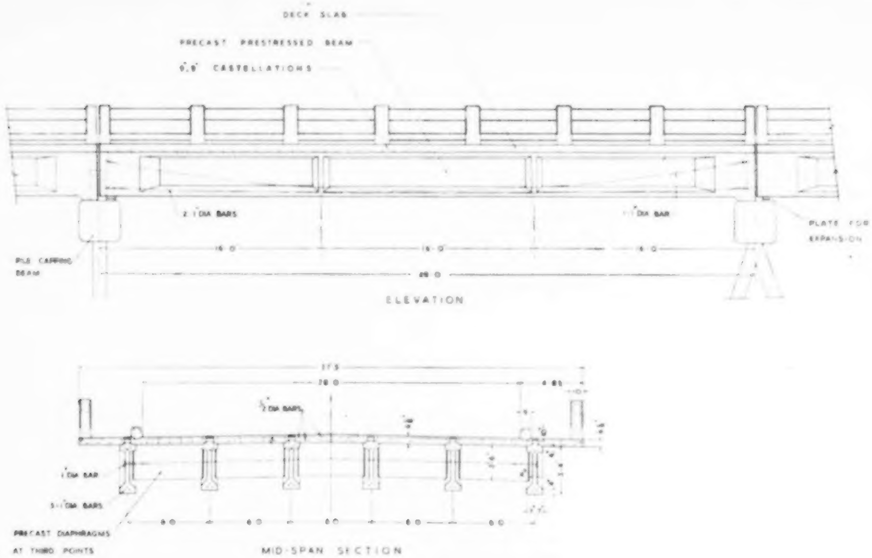


Fig. 2.

and $7\frac{1}{2}$ in. thick in the reinforced concrete bridge.

The design in both cases allowed for H20—S16 highway loading which, for spans of 36 ft. and 48 ft., is not much different from the Ministry of Transport highway loading in Great Britain. Every third trestle has raking piles to provide longitudinal stiffness. The use of prestressed concrete made a great economy in the quantity of steel required in the beams, and is also expected to make a substantial economy in the cost of the shuttering and handling. The economy in the total weight of steel is, however, not large, mainly because both designs have reinforced concrete deck slabs. The quantities are 350 lb. per linear foot for reinforced concrete and 230 lb. of mild steel plus 60 lb. of high-tensile bars and anchorages for prestressed concrete.

There are 363 spans, for which 2178 beams are being made. As the beams are made and stressed at the contractor's yard and transported to the bridge, and, while the deck slab is being concreted, they support the weight of the wet concrete, the design allows for four conditions of stress. Details of the beams are given in *Figs.* 2 and 3, and a detail of a joint over a trestle is given in *Fig.* 4.

The beams are cast in steel moulds. Some of the ducts are formed with removable pneumatic rubber tubes and some with flexible metal ducts. The metal tubes are left in the beams and the bar is placed in it before concreting is commenced. In both cases the space between the bar and the duct is grouted.

The concrete was specified to have a minimum 28-days' cylinder crushing strength of 5000 lb. per square inch, which corresponds roughly with a cube strength of 6000 lb. per square inch. The maximum compressive stress at release of

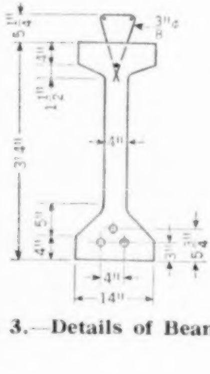


Fig. 3.—Details of Beam.

the prestressing force is 1710 lb. per square inch, and under the design load plus impact the compressive stress is 738 lb. per square inch.

A test of a beam showed that four times the maximum calculated shearing stress due to the design live load produced no cracks. The same beam tested with an overload in bending and without the deck slab cracked first under a central concentrated load of 40,000 lb., when the deflection was ½ in.; at a load of 60,000 lb. the deflection was 1·6 in., and on unloading the beam the residual deflection was 0·19 in. When the load was reapplied the deflections were similar to those in the first test, and the beam failed under a load of 75,000 lb.

Another beam was tested without the deck slab; in this case the ducts were formed by metal tubes in which the bars were placed before concreting. With a concentrated central load of 60,000 lb., the deflection was 1 in.; one bar was then disconnected and the beam failed on reloading to 60,000 lb. with a deflection of 3·3 in. As the beams were designed to act in conjunction with the deck slab they were necessarily over-prestressed as separate beams, and the test results are considered to be very good.

The next beam tested had a section of deck slab 6 ft. wide cast on it. This beam showed first cracks under a central concentrated load of 40,000 lb., and after loading to 76,000 lb. it returned to its original level. On reloading a third time failure took place at 97,000 lb., which is equivalent to the weight of the beam and the road slab plus five times the specified live load. The contractors are the Hardaway Contracting Company.

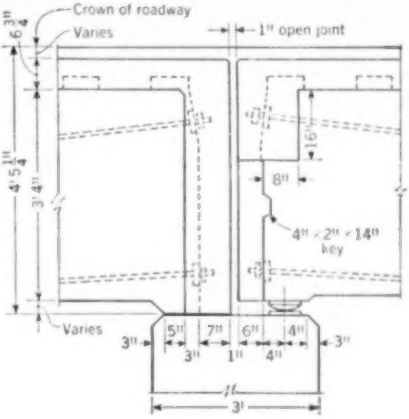


Fig. 4.—Details at Joint.

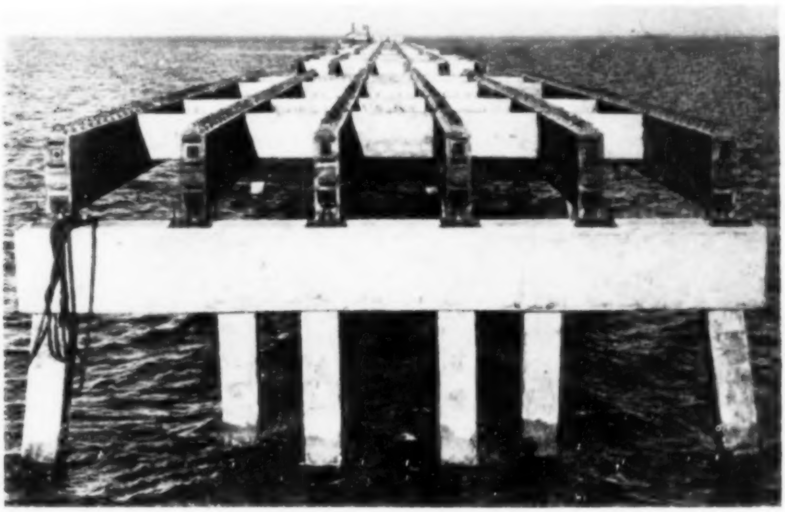


Fig. 5.—Beams and Diaphragms in Position.

Stranded-wire Cables for Prestressed Beams.

THE first floor of a garage recently completed in San Francisco, U.S.A., and described in "Engineering News-Record" for November 13, 1952, is supported by prestressed beams which are stated to be the largest yet made in the U.S.A. The beams comprise freely-supported spans of 60 ft. 6 in., and a two-span continuous beam with spans of 25 ft. (*Fig.* 1).

The freely-supported beams are 7 ft. 8 in. deep and are T-shaped with an upper flange 8 ft. wide by 1 ft. 4 in. deep. Each considered that the final force in each cable is 162,500 lb. The beam is designed to have zero stress at the upper surface of the top flange when carrying the dead load of the upper stories; consequently it was necessary to tension the cables after the upper stories were built. To avoid overstressing the columns as the beam deflected upwards, the columns above and below the beam are of steel pipes around which concrete was cast after the prestressing had been done.

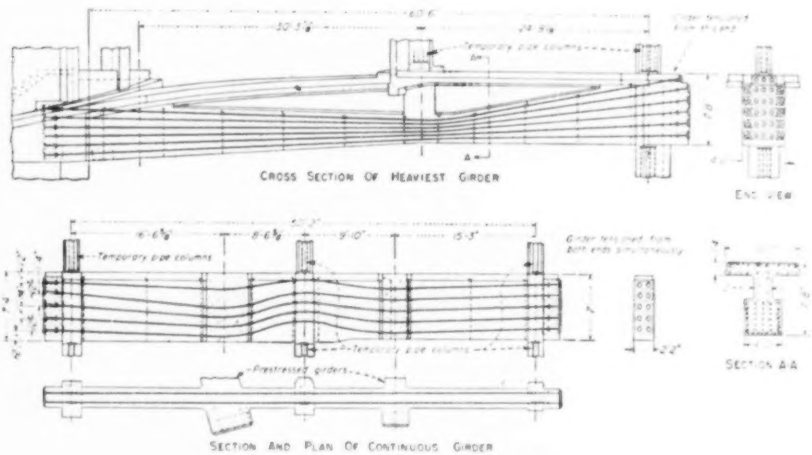


Fig. 1.—Details of the Beams.

beam supports a total load of 640 tons, of which 420 tons are concentrated at the centre of the span and are due to a column carrying the upper floors. The beams contain 28 cables of stranded wire; each cable is $1\frac{1}{2}$ in. diameter, with an area of 1·36 sq. in. and an allowable stress of 125,000 lb. per square inch. To prevent bond with the concrete the cables were greased and wrapped in thick building paper. The cables were anchored after tensioning by tightening a nut on threaded swaged fittings. In order to allow for loss of prestressing force due to the shortening of the beam during tensioning, the first cable was tensioned to 170,000 lb. and the force was reduced for each cable after, the last one being tensioned to 162,500 lb. The reduction in the length of the beam during tensioning was $\frac{3}{16}$ in., and it is

The advantage of prestressing the continuous beam is in the reduction of the principal tensile stress, which in a reinforced concrete beam of the same size would be 417 lb. per square inch; by prestressing the beam this is reduced to 221 lb. per square inch. The dimensions of this beam are given in *Fig.* 1. Ten $1\frac{1}{2}$-in. diameter cables are used and are curved down at the positions of the single-span beams and upwards at the central support. To reduce the losses due to friction the cables were tensioned from both ends.

The building was designed by Messrs. Ellison & King, consulting engineers, in collaboration with Mr. T. Y. Lin. Messrs. John A. Roebling's Sons Co. supplied the cables and jacks, and the general contractors were Messrs. Barrett & Hilp.

Design of Indeterminate Structures by the "Plastic" Method.

By R. GARTNER, D.Sc.

(Concluded from January, 1953.)

Rigid Frames.

The design of rigid frames can be made in a similar manner, but it must be remembered that systems of loading cannot be superimposed. Therefore different cases of total loads have to be examined, as is shown in the following examples.

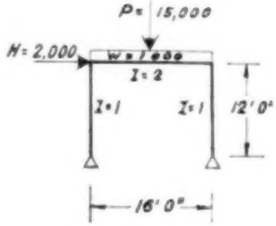


Fig. 8.

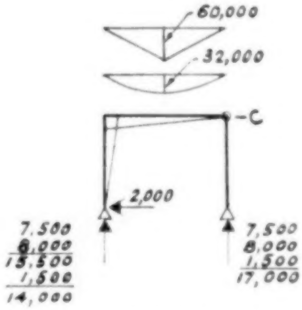


Fig. 9.

Example 5.—The two-hinged frame (*Fig.* 8) is to be designed. On account of the horizontal load the plastic hinge will develop at point C and the determinate case is a three-hinge frame with one hinge at C (*Fig.* 9). Case $X_1 = 1$ is shown in *Fig.* 10.

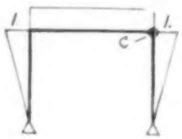


Fig. 10.

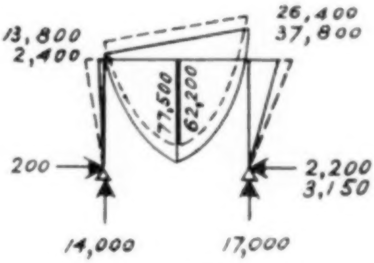


Fig. 11.

$$\delta_{11} = \frac{12}{3 \times 1} \times 2 + \frac{16}{2} \times 1 = 16.$$

$$\delta_{10} \text{ for } w = -\frac{2}{3} \times 32{,}000 \times \frac{16}{2} \times 1 = -171{,}000.$$

$$\delta_{10} \text{ ,, } P = -\frac{16}{2 \times 2} \times 60{,}000 = -240{,}000.$$

$$\delta_{10} \text{ ,, } H = -\left(\frac{12}{3} \times 24{,}000 + \frac{16}{2 \times 2} \times 24{,}000\right) = -192{,}000. \ \Sigma\delta_{10} = -603{,}000.$$

The equation for X_1 is $16X_1 - 603{,}000 = \delta_1$.

The plastic moment is $X_1 = M_C = \dfrac{0{\cdot}7 \times 603{,}000}{16} = 26{,}500$, while the elastic moment is X_1 elastic $= M_C = \dfrac{603{,}000}{16} = 37{,}800$. *Fig.* 11 shows the final moment lines for the two methods (full lines for the plastic method; dotted lines for the elastic method).

Example 6.—Calculate the rigid frame shown in *Fig.* 12 (columns of constant section). Two plastic hinges can be expected at points B and C, with a partial

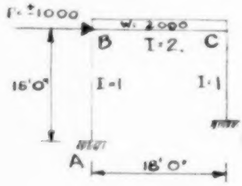


Fig. 12.

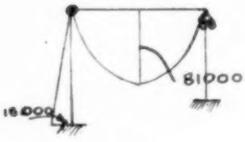


Fig. 13.

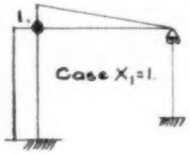


Fig. 14.

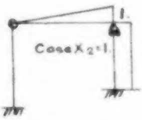


Fig. 15.

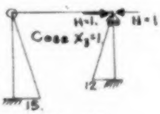


Fig. 16.

failure, when a third plastic hinge will develop in beam B–C. The determinate case is shown in *Fig.* 13. Case $X_1 = M_B = 1$ is shown in *Fig.* 14, Case $X_2 = M_C = 1$ in *Fig.* 15, and Case $X_3 = H = 1$ (at point C) in *Fig.* 16.

$$\delta_{11} = 15 + \frac{18}{3 \times 2} = 18\,; \quad \delta_{12} = \frac{18}{6 \times 2} = 1{\cdot}5\,; \quad \delta_{13} = -\frac{15^2}{2} = -112{\cdot}5.$$

$$\delta_{22} = \frac{18}{3 \times 2} + 12 = 15\,; \quad \delta_{23} = -\frac{12^2}{2} = -72\,; \quad \delta_{33} = \frac{15^3}{3} + \frac{12^3}{3} = 1701.$$

For $w = 2000$, $\delta_{10} = \delta_{20} = -\dfrac{18}{3 \times 2} \times 81{,}000 = -243{,}000\,;\ \delta_{30} = 0.$

For $P = 1000$, $\delta_{10} = \dfrac{15}{2} \times 15{,}000 = +112{,}500\,;\ \delta_{20} = 0.$

$$\delta_{30} = -\frac{15^2}{3} \times 15{,}000 = -1{,}120{,}000.$$

The equations for the unknowns for w only are

$$18X_1 + 1{\cdot}5X_2 - 112{\cdot}5X_3 - 243{,}000 = \delta_1 \quad . \quad . \quad . \quad (1)$$
$$1{\cdot}5X_1 + 15X_2 - 72X_3 - 243{,}000 = \delta_2 \quad . \quad . \quad . \quad (2)$$
$$-112{\cdot}5X_1 - 72X_2 + 1701X_3 = 0 \quad . \quad . \quad . \quad (3)$$

From the point of view of construction, the simplest solution is to make the two corner moments equal, that is $X_1 = X_2 = X$. From equation (3), $X_3 = \dfrac{184{\cdot}5}{1701}X$,

$X_3 = 0{\cdot}1085X$. Substituting in equation (1), $19{\cdot}5X - 12{\cdot}25X = 243{,}000$; $X = 33{,}500$. (2) $16{\cdot}5X - 7{\cdot}8X = 243{,}000$; $X = 28{,}000$.

Before deciding on the plastic moment the horizontal force P must be taken into account; for this force another method will be used.

$$\text{Column A–B has the stiffness } \frac{1}{15} = 0{\cdot}0666.$$

$$\text{,, \quad C–D \quad ,, \quad ,, \quad ,, } \quad \frac{1}{12} = \frac{0{\cdot}0833}{0{\cdot}15}.$$

Therefore the columns carry a load $P = 1000$ in the following ratio: Column A–B, $P_1 = \dfrac{0{\cdot}0666}{0{\cdot}15}1000 = 444$; column C–D, $P_2 = \dfrac{0{\cdot}0833}{0{\cdot}15}1000 = 556$. Approximating $M = 0$ in the middle of the columns, M left $= 444\dfrac{15}{2} = 3350$; M right $= 556\dfrac{12}{2} = 3336$.

Therefore, for $W + P$,

$M_{Bmax} = 33{,}500 + 3350 = 36{,}850$; $M_{Bmin} = 33{,}500 - 3350 = 30{,}150$; plastic moment $= 0{\cdot}7 \times 36{,}850 = 26{,}000$.

$M_{Cmax} = 28{,}000 + 3335 = 31{,}335$; $M_{Cmin} = 28{,}000 - 3335 = 24{,}665$; plastic moment $= 0{\cdot}7 \times 31{,}335 = 22{,}000$.

The larger value, that is M plastic $= 26{,}000$ will be applied.

Case A.—$w = 2000$ lb. per foot only (no wind); $X_1 = X_2 = 26{,}000$; $X_3 = 0{\cdot}1085 \times 26{,}000 = 2850$. *Fig.* 17 gives the moment diagram.

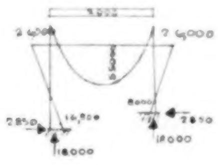


Fig. 17.

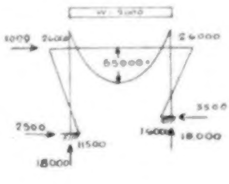


Fig. 18.

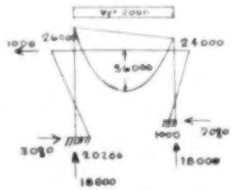


Fig. 19.

Check:

(1a) $18 \times 26{,}000 + 1{\cdot}5 \times 26{,}000 - 112{\cdot}5 \times 2850 - 243{,}000 = -56{,}000$
$= 23$ per cent.

(2a) $1{\cdot}5 \times 26{,}000 + 15 \times 26{,}000 - 72 \times 2850 - 243{,}000 = -20{,}000$
$= 8$ per cent.

Case B.—$w = 2000 + P = 1000$ lb. per foot (wind from the left). Again $X_1 = X_2 = 26{,}000$ (as $M_{Bmin} = 30{,}150$ and $M_{Cmax} = 31{,}335$). From (3), $-(112{\cdot}5 + 72)\ 26{,}000 + 1701X_3 - 1{,}120{,}000 = 0$. $X_3 = 3500$. *Fig.* 18 gives the moment diagram.

Check:

(1), $19{\cdot}5 \times 26{,}000 - 112{\cdot}5 \times 3500 - (243{,}000 - 112{,}500) = 17{,}500$
$= 13{\cdot}4$ per cent.

(2), $16{\cdot}5 \times 26{,}000 - 72 \times 3500 - 243{,}000 = -67{,}000 = 27{\cdot}5$ per cent.

Case C. (*Fig.* 19)—$w = 2000 + P = -1000$ lb. per foot (wind from the right). As $M_C = 24{,}665$, this value has to be used. To show what happens when a wrong value is used, M_C as well as M_B will first be assumed to be 26,000. From equation (3),

$$-(112{\cdot}5 + 72)26{,}000 + 1701X_3 + 1{,}120{,}000 = 0. \quad X_3 = 2150.$$

Check :

(1), $19{\cdot}5 \times 26{,}000 - 112{\cdot}5 \times 2150 - (243{,}000 + 112{,}500) = -91{,}500$
$= 26$ per cent.

(2), $16{\cdot}5 \times 26{,}000 - 72 \times 2150 - 243{,}000 = +32{,}000$, that is positive or of opposite sign to δ_{20}. This means that X_2 is too high.

Taking now the lower value $M_C = 24{,}000$, from (3),

$$-112{\cdot}5 \times 26{,}000 - 72 \times 24{,}000 + 1701X_3 + 1{,}120{,}000 = 0. \quad X_3 = 2080.$$

Check :

(1), $18 \times 26{,}000 + 1{\cdot}5 \times 24{,}000 - 112{\cdot}5 \times 2080 - (243{,}000 + 112{,}500)$
$= -90{,}000$ . . $25{\cdot}3$ per cent.

(2), $1{\cdot}5 \times 26{,}000 + 15 \times 24{,}000 - 72 \times 2080 - 243{,}000 = +6000$. Therefore there is a small positive value of $2{\cdot}5$ per cent. of δ_{20}. This is acceptable for all practical purposes. *Figs.* 17 to 19 show the moment lines for Cases A, B, and C.

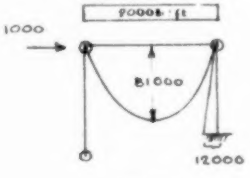


Fig. 20.

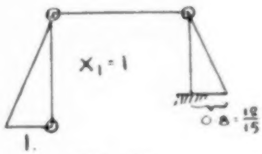


Fig. 21.

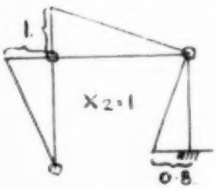


Fig. 22.

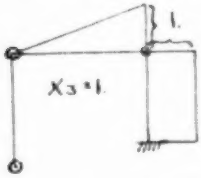


Fig. 23.

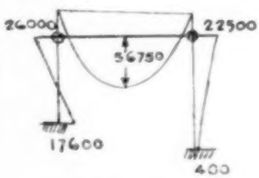


Fig. 24.

Example 6a.—The same frame as in *Fig.* 12, Case C, is to be designed with three hinges B, C, and D, where D is a steel hinge. *Fig.* 20 is the determinate case. *Figs.* 21 to 23 shows cases $X_1 = 1$, $X_2 = 1$, and $X_3 = 1$.

$$\delta_{11} = \frac{15}{3} + \frac{12}{3} \times 0{\cdot}8^2 = 7{\cdot}56. \quad \delta_{12} = \frac{15}{6} - \frac{12}{3} \times 0{\cdot}8^2 = -0{\cdot}06.$$

$$\delta_{13} = \frac{12}{2} \times 0{\cdot}8 = 4{\cdot}8. \quad \delta_{22} = \frac{15}{3} + \frac{18}{3 \times 2} + \frac{12}{3} \times 0{\cdot}8^2 = 10{\cdot}56.$$

$$\delta_{23} = \frac{18}{6 \times 2} - \frac{12}{2} \times 0{\cdot}8 = -3{\cdot}3. \quad \delta_{33} = \frac{18}{3 \times 2} + 12 = 15.$$

$$\delta w_{10} = 0\,;\ \delta w_{20} = \delta w_{30} = -\frac{2}{3} \times 18 \times 81{,}000\,\frac{1}{2} \times \frac{1}{2} = -243{,}000.$$

$$\delta P_{10} = \frac{12}{3} \times 0{\cdot}8 \times 12{,}000 = 38{,}400.$$

$$\delta P_{20} = -\delta P_{10} = -38{,}400\,;\ \delta P_{30} = \frac{12}{2}\,12{,}000 = 72{,}000.$$

The simultaneous equations are:

(1) $7{\cdot}56X_1 - 0{\cdot}06X_2 + 4{\cdot}8X_3 + 38{,}400 = \delta_1$.

(2) $0{\cdot}06X_1 + 10{\cdot}56X_2 - 3{\cdot}3X_3 - (243{,}000 + 38{,}400) = \delta_2$.

(3) $4{\cdot}8X_1 - 3{\cdot}3X_2 + 15X_3 - (243{,}000 - 72{,}000) = \delta_3$.

Checking the values from Example 6, namely, $X_1 = -20{,}200$; $X_2 = 26{,}000$; $X_3 = 24{,}000$:

(1*a*), $-152{,}500 - 1560 + 115{,}000 + 38{,}400 = \delta_1 = -660$ (approximately nil).
(2*a*), $+1200 + 274{,}000 - 72{,}500 - 281{,}400 = \delta_2 = 78{,}700$ (about 30 per cent.).
(3*a*), $-96{,}000 - 85{,}600 + 360{,}000 - 171{,}000 = \delta_3 + 7400$ (about $-4{\cdot}3$ per cent.).

These values of δ are slightly different from those in Example 6, but they are exact enough for slide-rule calculation.

To obtain the plastic value for X_1, equation (1) is used and, using $X_2 = 26{,}000$ and $X_3 = 22{,}500$ (which is smaller because equation (3) gave a positive value for δ_3), equation (1) gives $7{\cdot}56X_1 - 1560 + 107{,}500 + 38{,}400 = 0{\cdot}3 \times 38{,}400$ (30 per cent.). $X_1 = -17{,}600$.

Check: Equations (2) and (3):

(2), $1055 + 274{,}000 - 74{,}000 - 281{,}400 = -80{,}345 = 28{\cdot}6$ per cent.
(3), $-84{,}400 - 85{,}500 + 337{,}100 - 171{,}000 = -3900 = 2{\cdot}25$ per cent.

$X_3 = 22{,}500$ is the elastic moment, as the safe plastic moment (see Example 6) is 26,000. Therefore δ_3 should be nil, but a small value such as $2{\cdot}25$ per cent. is acceptable. *Fig.* 24 gives the bending-moment diagram.

Example 7.—Design the frame shown in *Fig.* 25 having a constant value of I (this example is considered elastically elsewhere[3]). The five-story frame is loaded horizontally only at joints C, E, and F. As in the case of vertical loading, where the free-beam moment measured from the base-line is unaltered, for horizontal loading at the joints the sum of the four corner moments of each story must be equal to the total horizontal moment of the story. The corner moments of the horizontal beams are the sum of the column moments above and below the joint and therefore represent the maxima, so that plastic hinges may develop in these joints. An adjustment for plasticity therefore affects two stories, so that it is better, for horizontal loading alone, to make the plastic adjustment as small as possible. Nevertheless it is not necessary to calculate such a frame exactly with many indeterminate unknowns, as a small adjustment may be allowed. The following is a quick method for such a calculation without using the equation for the unknowns.

As the frame is symmetrical about the vertical axis, the centre-points of the

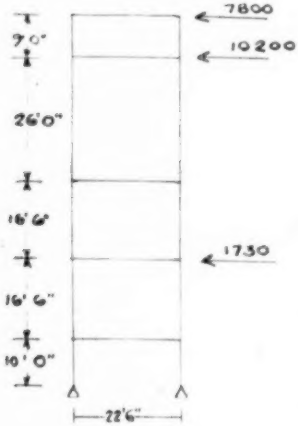


Fig. 25.

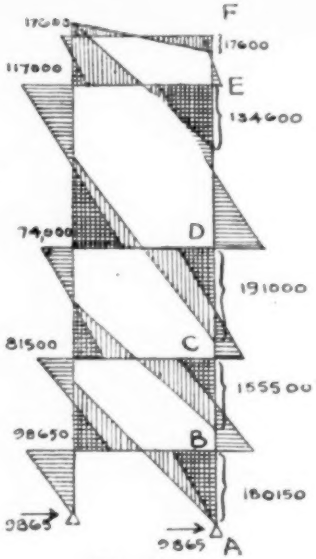


Fig. 26.

beams are points of zero bending moment (*Fig.* 26). Taking the middle points of the columns also as points of zero bending moment as a first approximation, the moment-line can readily be constructed (*Fig.* 26). The corner moments are:

$$\text{Story E–F,}\quad M_C = \frac{9}{4} \times 7800 = 17{,}600.$$

$$\text{D–E,}\quad M_C = \frac{26}{4} \times (7800 + 10{,}200) = 117{,}000.$$

$$\text{C–D,}\quad M_C = \frac{16{\cdot}5}{4} \times 18{,}000 = 74{,}000.$$

$$\text{B–C,}\quad M_C = \frac{16{\cdot}5}{4} \times (18{,}000 + 1730) = 81{,}500.$$

$$\text{A–B (hinged at A),}\quad M_C = \frac{10}{2} \times 19{,}730 = 98{,}650.$$

The beam moments are: $M_F = 17{,}600$. $M_E = 17{,}600 + 117{,}000 = 134{,}600$. $M_D = 117{,}000 + 74{,}000 = 191{,}000$. $M_C = 74{,}000 + 81{,}500 = 155{,}500$. $M_B = 81{,}500 + 98{,}650 = 180{,}150$.

The moments on the columns A–B are already exact, as the horizontal force on each column is $\frac{\Sigma H}{2} = \frac{19{,}730}{2} = 9865$.

Any other story can be considered as a closed frame by making cuts above the top and below the bottom corner and considering the internal forces of these cuts as external forces. Each of these closed frames is three times indeterminate

and, according to Mohr's rule[3], three checks can be used for the moment-line, namely, $\sum \frac{M \Delta s}{EI} = 0$, $\sum \frac{M x \Delta s}{EI} = 0$, and $\sum \frac{M y \Delta s}{EI} = 0$.

Fig. 26 shows that check (1) is fulfilled in all stories, as the same moments are positive on one side and negative on the other, and this also applies to check (2).

Check (3).—Taking the left column-line as the vertical axis, the static moment $S = \frac{\Sigma M y \Delta s}{EI}$ must not necessarily be zero (EI can be discarded as it is constant): *Fig.* 27 shows one story (note that the outside moments will be assumed to be negative, and the inside moments positive).

$$S_0 = \Sigma M y \Delta s = (-M_1 + M_2) \times \frac{l}{4} \times \left(\frac{5}{6} l - \frac{1}{6} l\right) = (-M_1 + M_2)\frac{l^2}{6}.$$

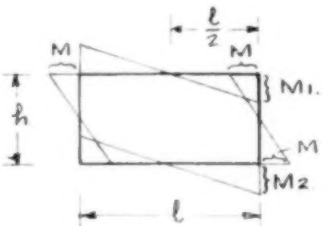


Fig. 27.

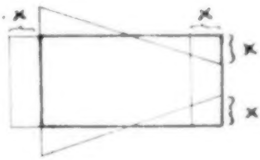


Fig. 28.

If S_0 is not zero the moment diagram is not exact and has to be adjusted. As the sum of the corner moments must equal the total horizontal moment, the diagram can be adjusted only as shown in *Fig.* 28 (or with the same diagram with opposite signs). Assuming, for the adjustment of a corner moment, $X = 1000$, then $S_X = (1000 + 1000)\frac{l^2}{6} + 1000\, h_1 l$.

The calculation is best computed in tabular form (see *Table* 1). First, the adjustment $X_1 = \frac{S_2}{S_1}$ is calculated. Secondly, the moment-line of the adjustment X_1 produces a new static moment S_{x1}. Thirdly, S_{x1} produces a second adjustment $X_2 = \frac{S_{x1}}{S_1}$, and so on. The series converges rapidly and can be stopped as desired for plasticity. The whole adjustment without plasticity is $\Sigma X = X_1 + X_2 + \ldots$ and this gives values (as a check) near enough to the values of the elastic equation (3). *Fig.* 29 gives the final moment diagram.

Example 8.—Design a frame as in *Fig.* 30 (which is also considered elastically elsewhere[3]). Although in this example the unknowns were not originally shown with a view to illustrating plastic design, it will be shown here that the plastic method can also be applied. The deformation values δ_{ik} were evaluated [3], and the simultaneous equations for the unknowns are as follows:

$$7622X_1 + 3384X_2 + 0 + 162{\cdot}88X_4 - 8{,}464{,}000 + \delta_1 \quad . \quad . \quad (1)$$

$$3384X_1 + 6293X_2 + 3384X_3 + 128{\cdot}86X_4 + 128{\cdot}86X_5 - 30{,}500{,}000 = \delta_2 \, . \quad (2)$$

$$3384X_2 + 7622X_3 + 0 + 162{\cdot}886X_5 - 12{,}696{,}000 = \delta_3 \quad . \quad . \quad (3)$$

$$162{\cdot}886X_1 + 128{\cdot}86X_2 + 0 + 15{\cdot}293X_4 + 3{\cdot}073X_5 - 1{,}462{,}000 = \delta_4 \quad . \quad (4)$$
$$128{\cdot}86X_2 + 162{\cdot}886X_3 + 3{\cdot}073X_4 + 15{\cdot}293X_5 - 1{,}650{,}000 = \delta_5 \quad . \quad (5)$$

To get an idea of the approximate value that should be chosen for the unknown, the two outside frames and the inside frame are considered separately as two hinged frames by making cuts at appropriate points and considering as external forces the internal forces at the cuts of the inside frame.

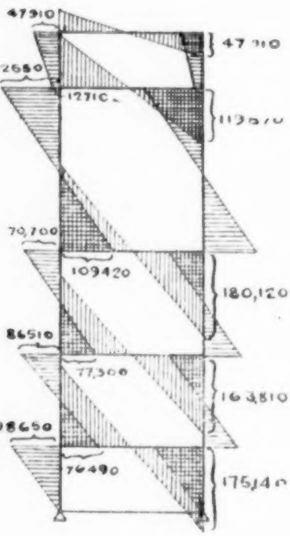


Fig. 29.

	A - B	B - C	C - D	D - E
S_0	−10,400	− 4800	+ 3050	− 206
S_x	372	755	541	348
X_1	+ 28	+ 6·4	− 5·6	+ 5·9
S_{x_1}	− 540	− 2850	− 1040	+ 472
X_2	+ 1·45	+ 3·8	+ 1·92	− 1·36
S_{x_2}	− 320	− 285	− 206	− 162
X_3	+ 0·86	+ 0·38	+ 0·38	+ 0·41
Σ_x	+ 30·31	+ 7·58	− 3·3	+ 5·01

All S-values are expressed in thousands

Table 1.

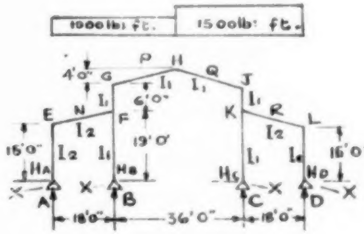


Fig. 30.

(a) Frame A–B.—Taking the moment at the middle of the beam EF as one-third of the free bending moment as a first approximation ($\frac{1}{3} \times 40{,}550 = 13{,}550$) the moment-line is established as in *Fig.* 31, and $H = \dfrac{40{,}550 - 13{,}550}{17} = 1600$ lb.

(b) Frame C–D.—In a similar way as a first approximation shown in *Fig.* 32, $M_{max.}$ positive = approx. $\dfrac{60{,}750}{3} = 20{,}250$, and $H = \dfrac{60{,}750 - 20{,}250}{17} = 2380$.

(c) Frame B–C.—This frame is loaded by the external forces and by the internal forces of frames A–B and C–D at the cuts (*Fig.* 33). The free moment at the middle at point H is 202,500 ft.-lb.[(3)]. Taking as a first approximation the maximum positive moment to be half the free moment, that is $\frac{1}{2} \times 202{,}500$ = say, 100,000 ft.-lb., the moment-diagram is as shown in *Fig.* 34 and

$$H = \frac{202{,}500 - 100{,}000}{29} = 3530 \text{ lb.}$$

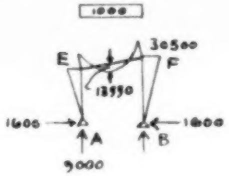


Fig. 31.

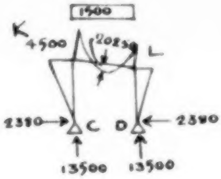


Fig. 32.

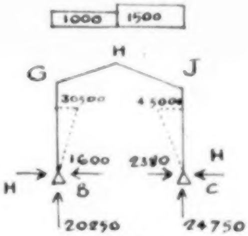


Fig. 33.

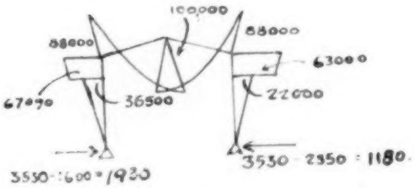


Fig. 34.

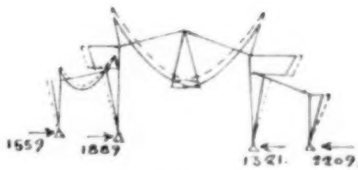


Fig. 35.

The unknowns are therefore, as a first approximation, $X_1 = -1930$, $X_2 = 3530$, $X_3 = -1150$, $X_4 = 67{,}000$, and $X_5 = 67{,}000$. Inserting these values in equations (1) to (5),

(1) $-7622 \times 1930 + 3384 \times 3530 + 162{\cdot}88 \times 67{,}000 - 8{,}464{,}000$
$= -314{,}000 = 3{\cdot}7$ per cent.

(2) $-3384 \times 1930 + 6293 \times 3530 - 3384 \times 1150 + 128{\cdot}86 \times 67{,}000 \times 2 - 30{,}500{,}000$
$= -1{,}660{,}000 = 5{\cdot}5$ per cent.

(3) $3384 \times 3530 - 7622 \times 1150 + 162{\cdot}886 \times 67{,}000 - 12{,}696{,}000$
$= +1{,}304{,}000 = -10{\cdot}2$ per cent.

(4) $-162{\cdot}886 \times 1930 + 128{\cdot}86 \times 3530 + (15{\cdot}293 + 3{\cdot}073) \times 67{,}000 - 1{,}462{,}000$
$= -72{,}000 = 4{\cdot}9$ per cent.

(5) $128{\cdot}86 \times 3530 - 162{\cdot}886 \times 1150 + (3{\cdot}073 + 15{\cdot}293)\ 67{,}000 - 1{,}650{,}000$
$= -134{,}000 = 8{\cdot}1$ per cent.

Examining the values δ_1 to δ_5, the following can be noted. (a) δ_1 and δ_3 should be zero as points A and B cannot move. (b) δ_3 is positive and therefore of opposite sign to δ_{03}. Modifying (a), if X_1 is decreased $-\dfrac{314{,}000}{7622} = -41$, δ_1 becomes nil; modifying (b), if X_3 is increased $\dfrac{1{,}304{,}000}{7622} = +171$, δ_3 becomes zero. Therefore, $X_1 = -(1930 - 41) = -1889$ and $X_3 = -(1150 + 171) = -1321$, and the values of δ will alter as follows:

$\delta_1 = -314{,}000 + 314{,}000 = 0$ (nil);
$\delta_2 = -1{,}660{,}000 + 139{,}000 - 580{,}000 = -2{,}101{,}000$ (6·9 per cent.);
$\delta_3 = 1{,}304{,}000 - 1{,}304{,}000$ (nil);
$\delta_4 = -72{,}000 + 6700 = -65{,}300$ (4·5 per cent.);
$\delta_5 = -134{,}000 - 28{,}000 = -162{,}000$ (9·9 per cent.).

These values are acceptable, but δ_2 should also become nil. This would alter X_2 by 33 lb. only, and can be neglected. On the other hand, δ_4 and δ_5 could have values higher than 4·5 and 9·9 per cent. Bearing in mind that points F and K are not the real plastic hinges but that these will be at points G and J, it is better to leave these two values of δ small.

As already stated, the values of X were not chosen for design by the plastic method and it would be more satisfactory to choose other unknowns, mainly bending moments. However, the example shows that, even in such a case, the calculation is easy and amenable to plastic design. If the adjustments are greater they are best carried out in tabular form; they are easily computed, and the values required soon converge. *Fig.* 35 shows the final moment diagram with the elastic case shown in dotted lines.

Conclusion.

It will be seen that in some of the examples a partial failure only was mentioned. The question arises of the number of elastic hinges which can develop before failure; apparently the answer is as many as the structure is times indeterminate, since the hinges cause the different parts to be free from each other and to make them eventually determinate. There is, of course, the same limitation as in the choice of the ordinary unknowns of any indeterminate structure, namely that the system must remain stable. There is, however, another limitation, namely, if there is more than one span or one story, partial failure in one span or story alone is possible. Apart from this, partial failure means that not all the plastic hinges which are possible actually develop. For instance, in Example (6) only two plastic hinges were contemplated while the system was three times indeterminate; it is evident that in reinforced concrete it is always possible to take advantage of all the plastic hinges by having additional steel hinges as in Example (6a) if the sizes of the concrete members are governed by other circumstances, that is by restricting the area of steel at these points. The values of v for steel hinges will be even greater than 30 per cent. so long as there is no danger of undue cracking; this question must be solved by research.[2]

The foregoing examples show clearly that there is not one way only of plastic design, but that nearly every method of calculation of indeterminate structures for the elastic stage can be adapted to the calculation of plastic design. The calculation itself will be much easier, but this does not mean that the elastic calculation is superseded. On the contrary, only the designer who is thoroughly conversant with the elastic method will be able to take full advantage of the method. These advantages are: (a) Easier calculation, since innumerable values are permitted, instead of the exact solution of simultaneous equations. (b) The elimination of the peak points of the moment diagram, which sometimes make the construction clumsy. (c) The liberty to choose the most economical and acceptable construction by being able to place the plastic hinges more or less anywhere.

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(2) Professor A. L. L. Baker. "Recent Research in Reinforced Concrete and its Application to Design," Institution of Civil Engineers *Structural Paper* No. 26.
(3) R. Gartner. "Statically Indeterminate Structures." Second Edition. Concrete Publications, Ltd.

Prestressed Footbridge at Edinburgh.

THE footbridge shown in *Fig.* 1 is the first of its kind in Scotland and was erected during the year 1952 to connect the Carrick Knowe and Stenhouse districts of Edinburgh. It crosses the main railway lines to Glasgow and Aberdeen with a clear span of 76 ft., and comprises precast concrete members forming two main beams at 8-ft. centres. Each beam is 6 ft. 2 in. high and consists of seven intermediate panels about 10 ft. long and two shorter panels at the ends. Between the panels are stiffeners, 6 in. wide, which connect the beams at the level of the bottom flange. Smoke baffles extend under the five central panels. *Fig.* 2 shows a typical panel and stiffener and cross sections through the bridge. The blocks at the ends of the beams are 10 in. thick (*Fig.* 2) and in them are cast anchor cones in recesses 5 in. square subsequently filled with mortar. The deck comprises precast hollow members spanning between the beams.

The bridge members were consolidated by vibration in timber moulds. The specified minimum crushing strength of the concrete was 6000 lb. per square inch at 28 days based on the average strength of samples taken from three different parts of each member. The mixture was 112 lb. of cement, 193 lb. of sand, 155 lb. of ¾-in. granite, and 155 lb. of ½-in. granite; the water-cement ratio was 0·41. Ducts were formed in each beam for five parabolic cables, each consisting of twelve wires of 0·2 in. diameter having a minimum 0·1 per cent. proof stress of 70 tons per square inch and an ultimate tensile strength of 100 to 110 tons per square inch. The panels and stiffeners contain a small amount of reinforcement mainly to resist stresses during handling, and mesh reinforcement was placed in the end panels near the cones to distribute the stresses due to the anchors.

Abutments and wing walls are constructed of two leaves of precast hollow blocks filled with concrete (*Fig.* 3), the courses varying in depth from 1 ft. to 10 in. in a height of 17 ft. The thickness of the wall, over the length of the abutment, is 5 ft. at the foundation and 4 ft. at the top. The bottom of the abutment and wing walls is shaped in plan to a radius of 585 ft., sloping at a batter of 1 in 17 to a straight line at the level of the coping.

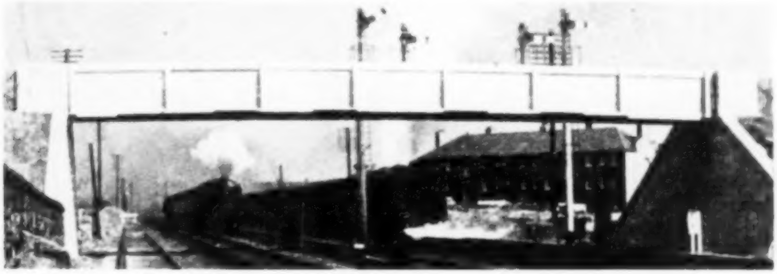

Fig. 1.

The precast members were transported by road to the site, where the bridge was assembled, with a camber of 6 in., on a temporary sleeper platform at footpath level and the joints caulked with cement-sand mortar before tensioning the cables. Each cable was tensioned to about 60 tons per square inch, producing an elongation of 5¼ in. Grout was then injected under pressure into the ducts.

After removal of the temporary supports the bridge was tested under a superimposed load of 112 lb. per square foot; the deflection was very small, with full recovery on removal of the load. The underside of the bridge was protected with bituminous emulsion over the width of the railway tracks. The beams were hoisted by British Railways in eight hours, using two 50-tons cranes. The

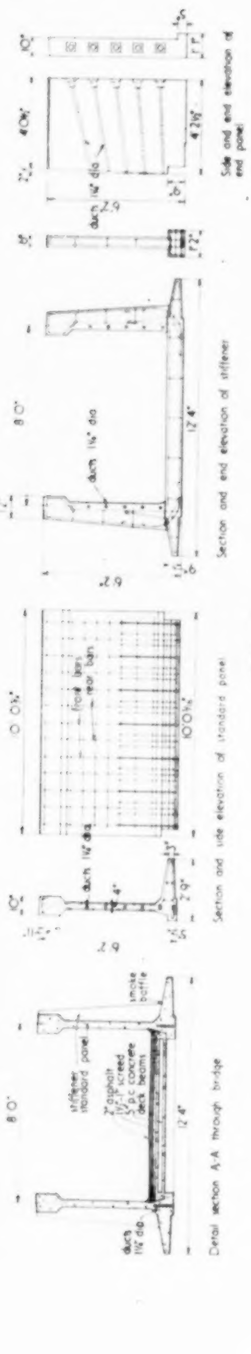

Fig. 2.

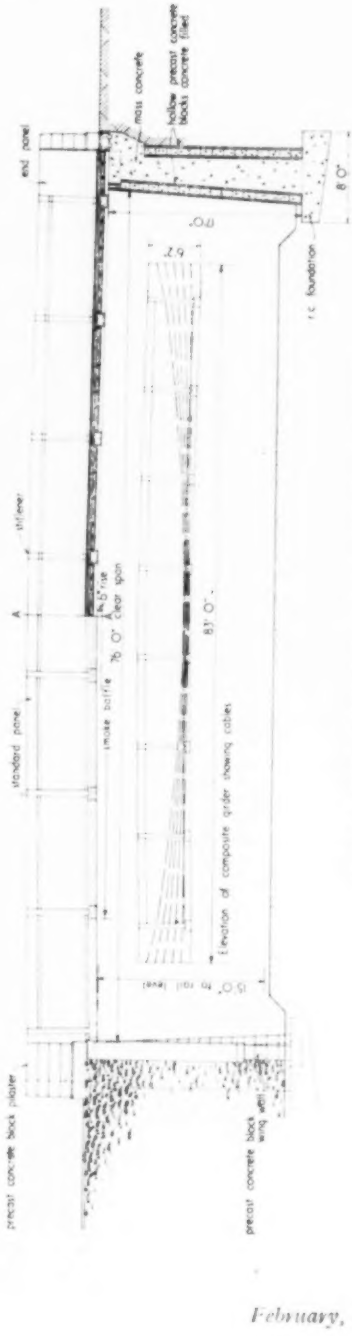

Fig. 3.—Part Elevation and Longitudinal Section.

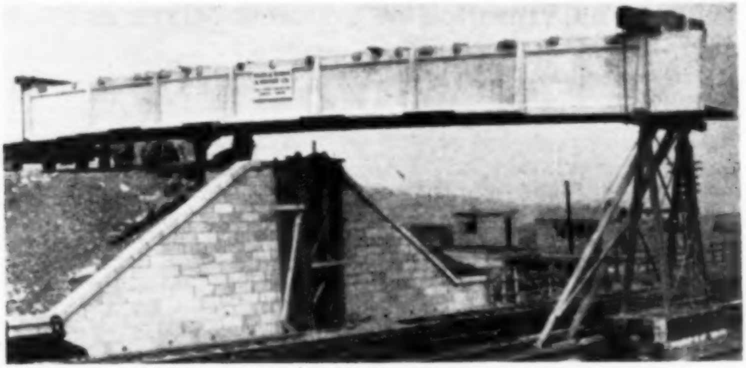

Fig. 4.—The Temporary Support at the Centre of the Tracks.

leading end of the bridge was first lifted on to a temporary trestle at midspan (*Fig.* 4). The cranes were then moved to the far track and from there moved the bridge to its final position. The leading end of the bridge was supported during lifting by a distributing beam (*Fig.* 5), the rear end of the bridge being supported on a bogey.

Precast granolithic padstones were provided at each abutment with three layers of bituminous sheeting under the end panels. Pilasters of precast blocks were built around the end panels, and the deck was completed by laying a screed over the precast members and finishing with an asphalt wearing surface.

The bridge was designed in the office of Mr. W. P. Haldane, M.B.E., B.Sc., M.I.C.E., City Engineer, Edinburgh, in conjunction with the Prestressed Concrete Co., Ltd., who supplied the equipment for tensioning the cables. The main contractors were Messrs. Melville, Dundas & Whitson, Ltd., and the precast members were made in the factory of the Springbank Quarry Co., Ltd.

Fig. 5.—Supporting the End of the Bridge during Lifting.

Cement Production in Great Britain in 1952.

For the fifth successive year total deliveries of cement in Great Britain for the home and export markets set up new records. The production for the home trade during the year 1952 increased by over one million tons, that is 12·3 per cent. greater than in 1951. The totals for the last three years are: 1950, 7,808,000 tons; 1951, 8,145,000 tons; 1952, 9,147,000 tons. Exports during the year totalled 2,055,000 tons, which is an increase of 4 per cent. over those for the year 1951.

In June, 1952, a reduction was made in the price of cement of 3*s*. a ton in most areas and 2*s*. a ton in other areas. Two further reductions of 2*s*. 9*d*. and 3*s*. a ton for packed cement were made later in the year owing to reductions in the price of paper.

An increase in the productive capacity of the industry of about 500,000 tons will be available in 1953, and a still larger increase in production is planned for the year 1954.

The Vibration of Concrete Mixtures.

We have received the following from Dr. J. M. Plowman of the Department of Civil Engineering, King's College, University of London.

"I would like to make one or two observations on the vibration of lean and gap-graded mixtures mentioned in your Editorial Notes in November in which a publication of the Ministry of Works is quoted to the effect that 'a reduction in the surface area of the aggregate used may be obtained by discontinuous or gap gradings in which certain sizes, usually those between $\frac{3}{16}$-in. and $\frac{3}{8}$-in. are omitted,' and that these mixtures produce somewhat harsher concretes but are suitable for compaction by vibration.

"Having studied the problem of vibrating these types of mixtures on tables for more than four years, I would like to utter a word of warning regarding the choice of grading for vibration. If the grading is unsuitable for vibration the mixture tends to rotate in the mould about a horizontal axis; air is sucked in between one side of the mould and the adjacent concrete, and is pumped into the body of the mixture thus preventing compaction. This effect is shown by a depression of the surface of the concrete at one side, accompanied by an elevation of the surface at the opposite side. This must not be confused with segregation, since the mass remains homogeneous. The loss in strength is due to the inclusion of air voids. To distinguish between segregation and this effect I have used the term 'rotational instability' when referring to the latter.

"The rotation appears to be caused by one portion of the mould having a slightly higher acceleration than the remainder. In a good mixture the internal resistance is such that rotation does not take place but if this internal resistance is too small then rotation occurs at a rate of several revolutions per minute. An increase in the proportion of sand between sieve sizes No. 52 and No. 100 appears to have a very large influence on the internal resistance, perhaps because the grains act in a similar manner to ball bearings between the larger stones.

"I have observed this effect on many occasions in precast works dealing with large masses of concrete. Invariably it has been referred to as segregation. In fact no segregation has taken place. This 'rotational instability' may cause a loss of strength of up to 50 per cent. of that obtained by vibrating a mixture with the same water-cement ratio which is stable.

"It is therefore necessary to be able to recognise this 'rotational instability' when it occurs and to be able to remove the cause. The idea that vibration will compact anything that cannot be compacted by hand must be qualified by requiring 'rotational stability' of the mixture."

Cables in Prestressed Concrete.

Further examination of the prestressed concrete footbridge at the Festival of Britain site on the South Bank, London (see this journal for July 1951 and June 1952) have, states the Cement & Concrete Association, shown that the grouting of the cables was unsatisfactory and in no case was the hole completely filled. In some cores no grout had been injected.

At midspan and over the supports, where maximum eccentricities were required, some movement of the rubber cores towards the centre had taken place, and the friction developed during tensioning was appreciable. The combination of these two factors may explain the presence of a crack near midspan at working load, since the stress produced in the concrete by the product of the force in the cable and the eccentricity of the cable was less than had been assumed in the design. It seems that very rigid fixing at close spacing is essential, so as to resist effectively the forces tending to straighten the core and avoid undulation due to the buoyancy of the tubes with consequent increased friction.

MISCELLANEOUS ADVERTISEMENTS.

Situations Wanted, 3d. a word: minimum 7s. 6d. Situations Vacant, 4d. a word: minimum 10s. Other miscellaneous advertisements, 4d. a word: 10s. minimum. Box number 1s. extra. The engagement of persons answering these advertisements is subject to the Notification of Vacancies Order, 1952.

Advertisements must reach this office by the 23rd of the month preceding publication.

SITUATIONS VACANT.

SITUATION VACANT. Reinforced concrete senior designer required for consulting engineer's office in London. Commencing salary £1,000 p.a. Apply Box 3623, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Structural engineer, office in London, has vacancy for designer-detailer to work under senior engineers in reinforced concrete. Must be thoroughly experienced in detailing, and some knowledge of design only. Salary £750–£850 p.a. according to experience. Details to Box 3624, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. A reinforced concrete consultant, Westminster, has vacancy for experienced detailer-draughtsman. Salary £450 to £650 p.a. Applications stating age and experience to Box 3625, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. Experienced reinforced concrete designers required for Bristol and Sunbury-on-Thames offices of reinforcement manufacturers. Positions offer opportunity to gain experience of wide range of structures. Attractive salaries offered to suitable applicants. Applications in confidence, stating qualifications and fullest details of experience, to Box 2609, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. Leading and senior civil engineering draughtsmen required with experience of reinforced concrete work and foundations for heavy industrial buildings and equipment. Also draughtsmen with experience of railway track layout. Permanent employment with good prospects to men with skill and initiative. Pension scheme in operation. Apply, stating age, experience, wage required, and when at liberty, to CHIEF TECHNICAL ENGINEER, CENTRAL ENGINEERING DEPARTMENT, DORMAN, LONG & CO., LTD., G.P.O. Box 11, Royal Exchange, Middlesbrough, Yorks.

SITUATION VACANT. Scaffolding, shuttering, builders' plant. Manufacturers wish to appoint qualified gentleman to organise and supervise the development of new lines in this field. Applicants with practical experience in reinforced concrete work, and good designer-draughtsman with knowledge of sales promotion, should apply to Box 3626, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. Draughtsmen required, fully conversant with steel formwork design. Applicants must be energetic and progressive and prepared to undertake occasional journeys overseas. The vacancy offers scope to men with ideas. Write fully, stating age and experience, etc., to HS/BP ACROW (ENGINEERS) LTD., 8 South Wharf, London, W.2.

SITUATION VACANT. Senior engineer required for consulting engineer's office, W.1 district. Considerable experience of reinforced concrete work essential, including precast and prestressed. Also good knowledge of general building construction. Experience of structural steelwork desirable. Responsible position. Applications in confidence, stating age and qualifications, with full details of experience, to Box 3629, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. Consulting engineers require several designer-draughtsmen with about 2–3 years' experience in reinforced concrete work. Salary in accordance with qualifications and experience. Applicants should be Graduates of either the Institution of Civil or Structural Engineers. Apply in writing to ANDREWS, KENT & STONE, 329 High Holborn, London, W.C.2.

SITUATION VACANT. Applications are invited for the post of reinforced concrete structural engineer to take charge also of concrete quality control department. First-class practical experience in structural design and construction is essential. Details of experience and salary required to REEMA CONSTRUCTION, LTD., Milford Manor, Salisbury, Wilts.

SITUATION VACANT. Senior structural engineer for consulting engineer's office in Liverpool required. Age 30–45. Salary up to £1,500 p.a., according to age and experience. Full details to Box 3630, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Consulting structural engineer, Westminster, requires senior designer-draughtsman with first-class experience in reinforced concrete for responsible position. Experience in structural steelwork an advantage. High salary and good prospects for suitable applicant. Write in confidence stating age, qualifications, and full details of experience. Box 3631, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. Bricklayers for erection of precast concrete cladding in Oxford area. Top rates, incentives, subsistence where applicable, and hostels near. ORLIT, LTD., Colnbrook By-Pass, Colnbrook, Bucks.

SITUATIONS VACANT. Costain Concrete Company require the services of two experienced reinforced concrete and hollow-block floor designers for a rapidly expanding programme. Reply in writing, giving full details of age, experience, and salary required, to PERSONNEL DIRECTOR, COSTAIN CONCRETE CO., LTD., Dolphin Square, London, S.W.1.

SITUATION VACANT. Designer-draughtsman required for London office of well-known reinforced concrete engineers. Experience in reinforced concrete frames, floors roof, and staircase construction essential. Progressive post, pension scheme, and alternate Saturdays free. Write fully to Box No. 204, ALLARDYCE PALMER, LTD., 109 Kingsway, London, W.C.2.

SITUATION VACANT. Costain Concrete Company require the services of an estimator/surveyor for their factory in Lanarkshire, Scotland. Must have had previous experience of flooring contracts, and in reinforced concrete. Apply in writing, giving full details of age, experience, and salary required, to the Manager, COSTAIN CONCRETE CO., LTD., Newmains Cross, near Wishaw, Lanarkshire, Scotland.

SITUATIONS VACANT. Designer-draughtsmen wanted for Westminster office of firm of consulting civil engineers engaged on industrial and marine undertakings chiefly in reinforced concrete. Salary according to age and experience. Non-contributory pension scheme in operation. Box 3627, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Experienced sales representative required by large company of manufacturers of concrete products for North-Eastern and Yorkshire area. First-class knowledge of concrete products, including cast stone and patented concrete floor beams, essential. Applicants must possess personality and drive, and be capable of making all necessary trade contacts. Applications invited, in confidence, from salesmen with the required qualifications residing in the territory. Give details of age, experience, sales record, and salaries received to Box 3628, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. The British Reinforced Concrete Engineering Co., Ltd., require several qualified designers and detailers with specialist experience for their Stafford, London, and Newcastle-upon-Tyne offices. Five-days' week and staff pension scheme. Apply to B.R.C. ENGINEERING CO., LTD., Stafford.

SITUATION VACANT. Design and development engineer required for large precast concrete factory. Knowledge of prestressed concrete an asset. A.M.I.Struct.E. preferred. Westminster office. Five-days' week. Write Box 958, c/o WALTER JUDD, LTD., 47 Gresham Street, London, E.C.2.

SITUATIONS VACANT. Clarke, Nicholls & Marcel, consulting engineers, require at their London office draughtsmen-detailers experienced in reinforced concrete. Permanent positions, interesting work. Apply in writing, stating age, experience and salary required, to 21 Westbourne Grove, London W.2.

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Book Reviews.

"Pile Foundations." By R. D. Chellis. (London: McGraw Hill Book Co. 1951. Price £5 6s. 6d.)

In 681 pages the design of piled foundations and the practice of pile driving are dealt with in considerable detail. The necessity for adequate sub-surface investigation is stressed, and the numerous methods of assessing the bearing capacity of single piles and groups of piles are discussed. The general dynamic formula presented is that due to Mr. Hiley. Both American and European practice is dealt with and many proprietary types of piles in timber, steel, and concrete are described in detail with the aid of numerous clear diagrams. A long chapter is concerned with the deterioration and preservation of piles, and a discussion of the causes of failure of piled foundations is given. Tables relating to hammers, extractors, steel bearing piles and sheet piles, soil data, and jetting equipment describe British as well as American plant. Appendixes include American specifications for timber, concrete, and steel piles, and preservative treatments for timber piles, together with the derivation of driving formulæ, numerical examples, comparative results of tests, and methods of designing precast concrete piles. The book concludes with an extensive bibliography.

"Architecture Préfabriquée." By P. Abraham. (Paris: Dunod. Price 960 francs.)

Based particularly on the author's experience in rebuilding parts of the war-damaged city of Orléans, this work describes, in the French language, methods of constructing single and multiple-story buildings of prefabricated elements. Two methods of building walls have been used. In one method a wall consists of two leaves of precast concrete slabs 2 ft. 7 in. by 2 ft. by 3 in. thick, and the space between them is filled with concrete to form a load-bearing wall. The slabs comprise 1 in. of concrete and 2 in. of pozzolanic mortar with high thermal insulating properties, and both leaves are bedded in cement mortar. The outer face of the wall is weather-proofed by glass fibre embedded in bitumen. In the other method three-sided precast concrete blocks of story height are placed vertically, the open sides fitting in grooves formed in the closed sides of adjacent blocks. Where necessary the blocks are filled with in-situ concrete, which may be reinforced.

The floors are generally of precast prestressed concrete ribs and clay tiles. The author describes the methods of construction in detail, and also gives details of the prefabricated bathroom and kitchen equipment used. The book is an interesting record of prefabricated construction applied to blocks of flats and schools.

"Soil Mechanics, Foundations, and Earth Structures." By Gregory P. Tschebotarsoff. (London: McGraw-Hill Book Company. Price 59s. 6d.)

This comprehensive book on foundations is based upon lectures given by the author at Princeton University in the United States. The book will appeal to students because of its clear presentation of the subject and to practising engineers because throughout the book theory is shown in relation to design and construction. The author emphasises the difficulties of analysing the stresses in soils because of the simplified assumptions that have to be made to allow a mathematical solution of the problem, and points out that the solutions provided by soil mechanics necessarily refer to idealised conditions. Consequently a thorough exploration of the site is necessary in order to determine the limiting conditions to which subsequent calculations can be related. In introducing the chapter on earth pressures the author shows the clear conception held by Coulomb as early as the year 1776 of the basic factors influencing the lateral pressure of soils. Many records of failures in foundations and of measurements taken on sites are included, and there is an extensive bibliography.

"Beton-Kalender. 1952." 2 vols. 41st edition. (Berlin: Wilhelm Ernst & Sohn. Price 16 D.M.)

This edition does not differ materially from the last. The chapters on the strength of materials and the theory of structures have been rewritten and fresh diagrams are included. In Volume II, sections have been added on elastic analyses of slabs, reinforced concrete pipes, and the planning and construction of roads. Sections on estimating and construction, scaffolding, and regulations of other countries have been omitted for revision. A proposed simplified method

of designing reinforced concrete members, first published in the previous edition, has been amplified by examples. The work of a German committee for unifying and simplifying methods of calculation of reinforced concrete has not been concluded but its preliminary findings are given in this book.

"Studies in Elastic Structures." By A. J. S. Pippard. (London: Edward Arnold & Co. 1952. Price 60s.)

THE twelve chapters of this book are separate studies of problems in the analysis of elastic structures. The problems differ widely and in many cases have arisen in connection with actual designs. Frequently in engineering it is not possible to make exact mathematical analyses, and the methods demonstrated for the solution of certain complex problems are of value in showing how results may be obtained with an accuracy sufficient for design. An interesting treatment of open-web girders is given in which it is assumed that the chords are replaced by a continuous member which can transmit to the booms only the same type of action as the chords. The method is applied to the analysis of building frames with lateral loading and to braced columns. A somewhat similar treatment is used to obtain the bending moments and deflections of interconnected girders in bridges. The analysis of multiple lattice frames, which receives scant attention in most text-books, is treated here in a clear manner. The book should be of considerable value to advanced students of engineering and to engineers concerned with the analysis of complex structures.

"Durchlaufträger." (Vol. II. Seventh edition.) By Adolf Kleinlogel and Arthur Haselbach. (Berlin: Wilhelm Ernst & Sohn. 1952. Price 46 D.M.)

MOST of this book deals with the calculation of bending moments on continuous beams with equal spans and moments of inertia, or with unequal spans and with moments of inertia such that the stiffnesses of the spans are equal. The most useful parts are the formulæ and tables for the determination of bending moments and shearing forces for various combinations of uniformly-distributed loads on the whole or part of a span, and for different arrangements of concentrated loads. Tables are included for beams with up to eight spans, and for beams having an infinite number of spans. Special attention is given to curves showing the maximum values of bending moments and shear forces where a combination of several loads is involved. Although these sections give many useful aids for saving labour and time, their application appears to be safe only in the hands of an experienced engineer.

A section is included on beams having varying moments of inertia, particularly beams with haunches, but its usefulness is reduced by the necessity to consult other books in order to obtain solutions. A novel presentation of the theory is given in the first section, where characteristic numbers are introduced in the determination of reduction and influence coefficients.

"Simple Examples of Reinforced Concrete Design." By Oscar Faber, C.B.E. (London: Oxford University Press. 1952. Price 9s. 6d.)

THE fourth edition of this well-known book has been largely rewritten to conform with the British Standard Code of Practice No. 114 and with Code of Practice No. 3, Chapter V, "Loading" (1952).

"Building Technician's Diary, 1953." (London: Association of Building Technicians. Price 5s. 6d.)

IN addition to the diary pages, this pocket-size book contains 112 pages of memoranda of use to those concerned with building, some pages of graph paper, and maps of Great Britain.

MISCELLANEOUS ADVERTISEMENTS.

(*Continued from p. lvi.*)

SITUATION VACANT. Engineering draughtsman required at Feltham, Middx., with commercial experience in reinforced concrete design, especially floors. Write details of experience, and salary required to Box 3634, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Young qualified detailer-designer with previous experience of reinforced concrete required in Birmingham. Some additional experience of structural steelwork and quantities an advantage. Write, giving full particulars of age, education, qualifications, experience, and salary required, to HENRY M. HALE & PARTNERS, CHARTERED STRUCTURAL ENGINEERS, 125 Edmund Street, Birmingham, 3.

SITUATIONS VACANT. Reinforced concrete designers and detailers required immediately. Knowledge of colliery structures desired but not essential. Experience of quantities an advantage. Staff pension and bonus schemes in operation. Box 3632, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Consulting engineers require experienced resident engineer for supervision of reinforced concrete water tower, Norfolk. Salary in accordance with experience. Immediate appointment. Apply in writing to J. C. MELLIS & CO., 110 Horseferry Road, London, S.W.1.

SITUATION VACANT. Consulting engineers require experienced resident engineer for supervision of extensive reinforced concrete warehouse, London area. Salary in accordance with experience. Immediate appointment. Apply in writing to J. C. MELLIS & CO., 110 Horseferry Road, London, S.W.1.

SITUATION VACANT. Sales manager required by large company in East Yorkshire. Preferably under 35 with comprehensive knowledge of sales, organisation, administration, and modern methods. Ability to control and direct representatives and initiate sales-promotion schemes. Applicants must have a first-class knowledge of concrete products, including cast stone and patent concrete floor beams—this qualification is essential, and no others need apply. Applications are invited in confidence, with details of age, experience, references, and salaries received. Box 3636, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Draughtsman (senior) required in London drawing office. Experience in layout work and the application of steel shuttering as used in the building and civil engineering trade would be an advantage. Excellent prospects of promotion at home or abroad within this rapidly-expanding Company. Write stating experience, age, and salary required. ACROW (ENGINEERS), LTD., South Wharf, London, W.2. Reference (JRT) Engineering Department.

SITUATION VACANT. Design and development draughtsman required by large precast concrete company, Westminster. Knowledge of prestressed concrete an asset. Ability to undertake calculations to H.N.C. standard essential. Post entails initial design and supervision of prototype products of widest range. Write Box 27, c/o WALTER JUDD, LTD., 47 Gresham Street, London, E.C.2.

SITUATION VACANT. Old-established Tees-side firm requires section-leader reinforced concrete designer-draughtsman, fully experienced in designing and detailing reinforced concrete structures, foundations, and other civil engineering work. Apply, giving full particulars and experience, quoting reference D, to Box 3635, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. MINISTRY OF WORKS.—Structural engineering draughtsmen required in London, at Risley (Warrington), Harwell (near Didcot, Berks.), and other centres. Accommodation available for single men at Risley and Harwell. London salaries—Draughtsmen: Up to £628 per annum. Starting pay according to age, qualifications, and experience. Leading draughtsmen: £623 to £733 per annum. Rates outside London slightly lower. Applicants should be experienced in design and/or detailing of reinforced concrete or structural steelwork. Work is varied and not confined to standard schemes. Although not established posts, many have long term possibilities; competitions are held periodically to fill established vacancies. Apply in writing stating

(*Continued on page 101.*)

MISCELLANEOUS ADVERTISEMENTS.

(*Continued from p. lvii.*)

age, nationality, experience, and locality preferred to CHIEF STRUCTURAL ENGINEER, W.G. 10/S.1, Ministry of Works, Abell House, John Islip Street, London, S.W.1.

SITUATION WANTED.

SITUATION WANTED. Civil engineer, 37, University degree, active personality, with executive abilities and experience in surveying road construction, steel structures, and work with reinforced concrete contractors specialising in industrial structures and heavy foundations, at present senior designer with reinforced concrete consulting engineers, in charge of section, working on own responsibility, attending to correspondence, site meetings, etc., desires suitable job, possibly as assistant to chief engineer. Location: London, but willing to travel, and specially interested in any West Indian or Canadian projects. Present salary £1,000 p.a., plus car allowance. Box 3633, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1

FOR SALE.

FOR SALE. Sacks, bags, and curing cloths for sale. You want the best type and quickest delivery. Write JOHN BRAYDON, LTD., 26 The Highway, London, E.1. Telephone: ROYal 1044.

FOR SALE. Warsop S.6 portable concrete breaker hammer for sale with single-cylinder engine. One drill. J. & F. EDWARDS, LTD., 359 Euston Road, London, N.W.1.

IMPERIAL COLLEGE OF SCIENCE AND TECHNOLOGY DEPARTMENT OF CIVIL ENGINEERING

Bursaries in Concrete Technology.

NOTICE IS HEREBY GIVEN that the election to Bursaries in Concrete Technology tenable as from October 1953, will take place in June, 1953.

Candidates must hold a degree in engineering at the time of taking up the award, and must also have a good knowledge of the theory of structures.

Bursaries are of the value of £350 per annum, out of which the College Tuition Fee has to be paid; the amount may be increased to £450 for those with industrial experience. In addition, one or two Senior Bursaries of £600 per annum may be awarded to outstanding men with a minimum of three years' experience in industry.

The course will be postgraduate and Bursars who successfully complete the course will be eligible for the award of the Diploma of the Imperial College (D.I.C.).

Applications must be received on or before June 1st, 1953, by the Deputy Registrar, City and Guilds College, Exhibition Road, London, S.W.7, who will, on written request, send full information and application forms.

FOR HIRE.

FOR HIRE. Lattice steel erection masts (light and heavy), 30 ft. to 150 ft. high, for immediate hire. BELLMAN'S, Terminal House, London, S.W.1. Telephone: Sloane 5259.

Lectures on Concrete.

THE following lectures have been arranged by the Ministry of Works. Admission is free.

Care and Maintenance of Builders' Plant, by H. E. Hodgson. Town Hall, Luton. February 11. 7.30 p.m.

Building on Shrinkable Clay, by A. D. M. Penman. Carnegie Hall, Abington Street, Northampton. February 11. 7.15 p.m.

Essentials of Good Concreting, by E. E. H. Bate. Willesden Technical College, Denzil Road, London, N.W.10. February 12. 7.30 p.m.

Formwork and Concreting, by S. White. Technical College, Lancaster. February 12. 7.15 p.m. Also Municipal College, Burnley. February 24. 7.15 p.m.

The Thermal Insulation of Buildings, by R. R. Houston. Crown and Anchor Hotel, Ipswich. February 16. 8 p.m. By J. M. Boardman. Hereford College of Further Education, Newton Road, Hereford. February 18. 7.15 p.m.

Floor Finishes, by W. J. Warlow. Wandsworth Technical College, Wandsworth High Street, London, S.W.18. February 18. 7.30 p.m.

Foundation Problems, by L. R. Creasy. Municipal College, Anglesea Road, Portsmouth. February 19. 7.15 p.m.

The R.I.B.A. Form of Contract, by R. W. Porter. Watford Technical College (Room 198), Hempstead Road, Watford. February 19. 7.0 p.m.

British Standards and Codes of Practice for Concrete, by C. Roland Woods. Broadway Hall, North Herts. Technical College, Letchworth. February 24. 7.30 p.m.

Problems of Plastering and Rendering, by E. L. E. Westbrook. Coventry Technical College, The Butts, Coventry. February 24. 7.15 p.m.

Settlement in Buildings, by S. Mackay. The Young People's Institute, George Street, Kingston-upon-Hull. February 25. 7.15 p.m.

Surface Finishes of Concrete, by J. G. Wilson. Ministry of Works Building, Ashley Street, Birmingham. February 26. 7.15 p.m.

DATA FOR PRICING REINFORCED CONCRETE.

Materials.

(Delivered in London area.)

AGGREGATES (per cu. yd.). — Washed sand, 21s. 9d. Clean shingle: ¾ in., 18s. 5d.; ⅜ in., 21s. Pit ballast, 20s. 11d.

CEMENT (per ton, delivered at Charing Cross).—
- Portland cement, 6 tons and upwards, 91s. 1 ton to 6 tons, 100s. Paper bags and non-returnable jute sacks included.
- Rapid-hardening Portland, 8s. above ordinary Portland.
- Aquacrete and 417, 32s. 6d. above ordinary Portland; paper bags included.
- Colorcrete (buff, red, and khaki), in 6-ton loads, 132s. 6d.; paper bags included.
- Snowcrete, £12 11s. 6d., inc. paper bags.
- "Super-Cement," 32s. 6d. per ton above ordinary Portland cement; paper bags included.
- High-alumina cement, 1 ton and upwards, 280s. per ton.
- Snowcem paint, 71s. per cwt. inc. containers.

SHUTTERING.—For prices of timber, refer to S.R. & O., 1949, No. 1079 (price 1s. 1d.) and No. 94 (price 5d.) issued by H.M. Stationery Office.

REINFORCEMENT.—Mild steel bars, B.S. 785 (per cwt.); ⅝ in. to 2⅜ in., 39s. 6d. 5/16 in. to ½ in., 41s. ¼ in., 41s. 6d. 3/16 in., 43s.

Materials and Labour.

(Contracts up to £5000. Inc. 10 per cent. profit.)

PORTLAND CEMENT CONCRETE, 1 : 2 : 4.—
- Foundations, 2s. 3½d. per cu. ft. Columns, 3s. 2d. per cu. ft. Beams, 2s. 11d. per cu. ft. Floor slabs 4 in. thick, 8s. 3½d. per sq. yd.; Do., 5 in., 10s. 5d.; Do., 6 in., 11s. 11d.; Do., 7 in., 13s. 11½d. Walls 6 in. thick, 13s. 4½d. per sq. yd. Add for hoisting above ground level 3s. 7d. per cu. yd. Add for rapid-hardening Portland cement 2s. 6d. per cu. yd.

REINFORCEMENT.—Mild steel bars (B.S. 785), including cutting, bending, fixing, and wire (per cwt.)—¼ in. to ⅜ in., 69s. 9d. 7/16 in. to ⅝ in., 64s. 11/16 in. to 2⅜ in., 58s. 6d.

SHUTTERING AND SUPPORTS.—
- Walls, 201s. per square.
- Floors (average 10 ft. high), 186s. 6d. per square. In small quantities, 2s. 5½d. per sq. ft.
- Columns, average 18 in. by 18 in. (per sq. ft.), 2s. 10d.; in narrow widths, 3s. 7d.
- Beam sides and soffits, average 9 in. by 12 in. (per sq. ft.), 2s. 9d.; in narrow widths, 3s. 3d.
- Raking, cutting, and waste, 5½d. per lin. ft.
- Labour on splays, 2½d. per lin. ft.
- Small fillets to form chamfers, 5½d. per lin. ft.

Wages.

The rates of wages on which the above prices are based are: Carpenters and joiners, 3s. 8d. per hour (carpenters 2d. a day tool money); Labourers, 3s. 2½d.; Men on mixers and hoists, 3s. 4½d.; Bar-benders, 3s. 5d.

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